INVESTIGATION OF CORROSION OF MSE WALLS IN NEVADA

Final Report to:
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Abstract

Nevada has over 150 mechanically stabilized earth (MSE) retaining walls at 39 locations. Recently, high levels of corrosion were observed due to accidental discovery at two of these locations, specifically I-515/Flamingo Road and I-15/Cheyenne Avenue intersections. The resulting investigations of these walls produced direct measurements regarding the corrosion losses of the soil reinforcements, which included both bare steel and galvanized steel and electrochemical properties of the MSE backfill in order to identify its aggressiveness. One of the three walls at the Flamingo intersection was replaced with a cast-in-place tie-back wall at great expense because of the significant metal loss due to corrosion. The initial Flamingo investigation focused on average uniform corrosion loss values from the direct reinforcement measurements and laboratory backfill test results based on a variety of test methods. The investigation results are reevaluated in this report, through the incorporation of statistical analysis in order to effectively undertake a prediction that includes the variability in electrochemical properties.

The investigation found that the original MSE backfill approval test results are significantly different from those measured in the subsequent investigations. A correlation has been developed between two distinctly different soil resistivity laboratory test methods, namely the Nevada T235B and AASHTO T-288 methods. The Nevada test method under predicts the corrosive nature of backfill soils when compared to the AASHTO test method. A Nevada test predicting mildly corrosive backfill would be

evaluated as corrosive using the AASHTO procedure. As the Flamingo and Cheyenne investigations show, this has proved detrimental to the service lives of MSE structures.

The internal stability analysis of the two remaining MSE walls at the Flamingo intersection were also analyzed using corrosion loss models developed from the statistical analysis of the direct measurements. The results of the analysis from these two intersections were subsequently extrapolated to other Nevada MSE walls. Through review of the backfill approval data, specific Nevada MSE walls have been ranked relative to estimated backfill aggressiveness and specific suggestions for future corrosion analysis are recommended. There are four groups of evaluation methods that have been identified in this research. Each of these methods has its own usefulness, but some will be more costly than others. The four groups of evaluation methods for existing walls include representative backfill soil testing, installation of non-stressed soil reinforcements, nondestructive monitoring methods, and destructive direct observational methods.

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Table of Contents

List of Tables

List of Figures

Chapter One
Introduction

One of the most cost effective earth retaining structures used in transportation applications around the United States is the mechanically stabilized earth wall system, which are commonly referred to as MSE walls. These wall systems are comprised of a wall facing, typically concrete, that is oriented in a near vertical to vertical direction (Figure 1). Behind the facing there is a soil mass that has reinforcement inclusions which stabilize the backfill soils and allow for vertical construction. Such walls are typically found in tight intersections where room for slopes is not available. The soil reinforcements provide tensile strength which is an ability that the soil does not have. The spacing of the soil reinforcements, or inclusions, is relatively close together, approximately two feet apart vertically. These soil reinforcements and their interaction with the soil create the internal stability of the wall.

In Nevada, over 150 MSE walls have been constructed using metal reinforcements. It is well documented that when metals are buried they can experience corrosion due to the electrochemical interaction with the soil. This also holds true for soil reinforcements used in MSE walls. Part of the design process involves adding extra steel cross sectional area, also referred to as sacrificial thickness, to account for metal loss due to corrosion. MSE backfill soils that are mild to non-corrosive only are allowed by specifying a series of pass/fail controls (specifications) in order to limit the amount of corrosion. Specific metal loss models have been developed from corrosion studies in order to quantify the sacrificial thickness estimates. When the combination of sacrificial thickness and mildly corrosive soils are used together, MSE walls are expected to perform as desired.

However, if adequate sacrificial thickness is not used, or an aggressive environment exists in the backfill there will be high rates of corrosion, which can directly affect the internal stability of an MSE wall. At two locations in Las Vegas, Nevada, MSE wall soil reinforcements were found to have high amounts of corrosion. These two locations include the three MSE walls at the I-515/ Flamingo intersection and one MSE wall at the I-15/Cheyenne intersection. The former wall reinforcement corrosion was found by accident during construction of a soundwall at the top of one wall. The later was also found by accident during demolition of a portion of an MSE wall for an expansion project.

The Flamingo intersection is of significant interest because the case study is well documented. In 2004, the reinforcements in the largest of the three walls were found to be so corroded that the Federal Highway Administration recommended the wall be mitigated. A cast-in-place tie-back wall was constructed in front of the existing MSE wall. Also during that time, McMahon & Mann Consulting Engineers (MMCE) were hired to investigate the corrosion of all three MSE walls at this intersection. Their investigation, under the direction of Dr. Kenneth Fishman, evaluated the corrosive nature of the backfill and collected and performed direct measurements of the soil reinforcements for the three MSE walls. From their analysis, uniform average corrosion loss rates were estimated. Stability analyses were also performed for the remaining two MSE walls at the intersection based on remaining reinforcement capacity.

The Flamingo MSE wall investigation led NDOT to wonder how many other MSE walls may be experiencing stability issues due to high rates of corrosion. The research presented in this report is focused on developing a systematic approach to answer this question. There are other MSE wall case studies that can be used to help in this research, such as the South African MSE wall case study and the Caltrans Mariposa MSE wall case study. However, the data collected by MMCE presents a detailed MSE wall evaluation within Nevada. While MMCE focused on a uniform corrosion loss evaluation, further analysis, which focuses on a statistical evaluation of loss measurements to predict future stability issues due to corrosion loss, is presented in this report. The two remaining unmitigated walls at Flamingo are the focus of the stability analysis because they have not been mitigated and possess the ability to cause disruption to the transportation corridor and potential loss of life if they fail. A statistical approach has also been used to evaluate the characteristics of the backfill sampled in 2005 from behind the MSE walls and make comparisons to the data from approved backfill sources prior to construction in 1985.

The results from the statistical analyses performed provide the framework to select other walls that may be experiencing similar rates of corrosion. A database of existing MSE walls has been developed in order to aide in the selection of suspect walls. Wall locations and characteristics of the walls at those locations have been collected and included in the database. From this database, walls can be ranked in order of perceived severity so that future MSE wall evaluations can be performed. Specific methods for future analysis have also been developed and presented in this report as well.

1.1 Project Information

This research is a result of previous investigations and measurements produced by several groups including Nevada Department of Transportation (NDOT) and McMahon & Mann Consulting Engineers at the three MSE walls located at the I-515/Flamingo Road intersection in Las Vegas, Nevada. A proposal for further investigation into the severity of corrosion of MSE wall reinforcements was proposed by Drs. Raj Siddharthan and Barbara Luke from the University of Nevada in Reno and Las Vegas, respectively. The scope of the proposal is outlined below. The research and results presented in this report use this scope as a framework to identify potential corrosion problems, quantify these problems, and make predictions about their potential to affect other MSE walls in Nevada.

1.1.1 Scope of Project

To identify the extent of the elevated levels of corrosion for walls across Nevada, a series of six tasks were defined in the proposal to NDOT. These tasks are as follows:

1. Develop an Inventory of NDOT MSE Walls and Literature Survey;
2. Synthesize Available Field Inspection Database on the Behavior of Nevada MSE Walls;
3. Review the Report Relative to the Flamingo MSE Walls Prepared by McMahon & Mann, Consulting Engineers;

4. Assemble Data on MSE Wall Corrosion Performance and Specifications from Other States;
5. Identify and Synthesize Data on Important Factors that Affect Corrosion of Nevada MSE Walls; and
6. Select Candidate Sites for Phase II Investigation.

1.2 Organization of the Report

This report is divided into seven chapters where the introduction and conclusions and recommendations are the first and last chapters. In Chapter 2 there is a thorough discussion of the history of MSE wall corrosion background. This background starts with a development of historical buried metal corrosion studies relevant to MSE walls. There are several important MSE wall case histories that are summarized because of their relevance. This chapter also gives historical background regarding the agencies that have developed specification guidelines for corrosion issues related to soil reinforcements in MSE walls. Chapter 3 focuses on the mechanisms of corrosion of buried metals and discusses the testing issues and methods used to identify corrosive backfills. In this chapter an important development of the correlation between the Nevada T325B and AASHTO T-288 soil resistivity test methods is developed.

In Chapter 4 two crucial MSE wall corrosion case studies that have been conducted in Nevada are discussed. These two case studies include MSE walls at the I-515/Flamingo intersection and the I-15/Cheyenne intersection. Statistical analysis of the reinforcement section loss due to corrosion is performed. Statistical analysis of the differences in electrochemical properties measured in the backfill approval process compared to the in-place sample properties is also presented. Wall stability analyses are conducted for two Flamingo MSE walls as well.

Chapter 5 and 6 include proactive discussion of other MSE walls in Nevada that may be experiencing corrosion. Chapter 5 focuses on a database of the NDOT MSE walls and their characteristics. Chapter 6 is a detailed discussion of which NDOT MSE walls should be investigated further. A suggested evaluation sequence and statistical sampling practice is also introduced.

The conclusions of the report and recommendations for Phase II work are included in Chapter 7. Recommendations for modification to current NDOT practices with respect to MSE wall corrosion are also discussed. The tables and figures referred to throughout the report are included after the text. The references follow the figures. There is an appendix (Appendix A) at the end of the report. In this appendix, sample calculations for the MSE wall stability analysis are included.

Chapter Two
Historical Background

In order to better understand the issues related to corrosion of MSE walls a summary of some historical background has been included in this chapter. Corrosion studies have been conducted on metals buried in soil. These represent the basis for the estimation of sacrificial steel thickness that needs to be added in order to account for the natural phenomenon of corrosion. While the inclusion of sacrificial steel has been successful with a number of walls located around the globe there are a number of walls that have been found to perform poorly. A summary of some of these walls has been included to highlight some of the issues that still remain. In recent years, because of the unknowns that still surround corrosion of MSE walls, surveys of MSE wall owners, typically state DOTs, have been conducted. Three of these surveys, which represent the most recent surveys, give an overall idea of the number of MSE walls that exist in the United States and some of the findings of walls that have faced corrosion issues. Finally, this chapter also identifies the specific historical recommendations and practices by FHWA, AASHTO, and Nevada.

2.1 Historical Corrosion Studies

While there has been a number of corrosion studies of metals buried in soil, there are two that stand out for MSE wall corrosion issues. These are the forty-five year study performed by the National Bureau of Standards and a study performed by a French laboratory in conjunction with the Reinforced Earth Company. The first is a general study of an assortment of metals in a variety of soil types and environmental conditions.

The latter is more specific to MSE walls focusing on soils and conditions that are more representative of MSE wall construction practices.

2.1.1 National Bureau of Standards Circular 579

In April 1957 the National Bureau of Standards (NBS, now the National Institute of Standards and Technology) released its NBS Circular 579 (Romanoff 1989). This circular was the result of a forty-five year study (commissioned in 1910) of underground corrosion of metals in different environments which is considered by many as the beginning of concentrated research on the effects different soil conditions on the corrosion of metals. One of the outcomes of this research is an understanding that pH, soil resistivity, and soluble salts, in conjunction with moisture content, affect the rate at which corrosion occurs. From this understanding engineers are able to estimate metal loss of buried metal structures, such as pipelines and, more important to this paper, the metal loss of soil reinforcements for MSE walls.

From this extensive study several concepts of the corrosive nature of soil became apparent. The development of the empirical relationship of time and metal loss (measured by pitting depth) is expressed as,

$$P=kt^n \tag{2.1}$$

where P is the pit depth at time t, and k and n are constants that depend on the soil and metal characteristics, respectively. It was typically seen that the corrosion rate was higher at original burial and tended to tapper off to a lower rate as time since burial continued. Much of the current practice in metal loss assumption stems from this study. However, it was widely understood that soils which were used in these tests were not entirely representative of the soils used in the typical construction of MSE walls. Specifically there were only a few sites where sands and gravels with low fines content and low plasticity were tested. Further studies were subsequently conducted in order that the constants in Equation 2.1 could be more representative of the backfill materials used in MSE wall construction.

2.1.2 Reinforced Earth Company Study

The Reinforced Earth Company along with the Laboratorie Central des Ponts et Chaussees (LCPC) and Terre Armee International realized the limitations of the NBS study with respect to MSE wall corrosion (Darbin et al. 1988). In 1974 these two groups combined efforts to study the effects of MSE backfill on soil reinforcements, more specifically buried galvanized and bare steel reinforcements. The reinforcements were tested in controlled soil boxes containing a variety of soil types with differing electrochemical characteristics. Also included in this study was the evaluation metal loss of in-service soil reinforcements of forty existing Reinforced Earth Company walls located in France.

The environmental controls of some of the tests included five soil types and a variety of water contents. Electrochemical effects of chlorides and sulfates were evaluated using one soil type and various levels of the soluble salts along with varying water contents. The reinforcements used in the tests included black steel (not galvanized) and steel that was galvanized, but with different thicknesses of zinc coatings.

In order to evaluate the metal loss of the samples due to corrosion two methods were used, which included container tests and electrochemical tests. With the container tests samples were accurately weighed, buried and exhumed at specific time intervals and then cleaned and weighed again. The difference in weight was then converted to a uniform metal loss. The samples that were measured electrochemically using polarization resistance measurements were then compared to the container tests, which showed that in most cases both predicted similar uniform metal loss rates. Although much of the data revolves around uniform loss, the authors pointed out that once the galvanized coating had been oxidized the underlying steel became pitted and it was confined to localized regions (Darbin et al. 1988).

After a ten year study of these buried reinforcements, conclusions were drawn as to the applicability of the NBS data and new design methods especially concerning buried steel with galvanized coatings. It was found that water content, sulfates and chlorides all played significant roles in the rate of corrosion of steel reinforcements within MSE backfill. More precise values for the constants for input into Equation 2.1 were also developed from the ten years of data based on the "practice to restrict extrapolations to a period lasting no more than ten times the duration of the actual measurements" on a logarithmic scale (Darbin et al. 1988, pg. 1031).

2.2 Historical Field Investigations

One of the primary and effective methods for learning about corrosion and its effects on soil reinforcements is to conduct field investigations of existing MSE walls. These types of investigations are very costly and in transportation corridors cause extra burden on the population that depend on their usefulness. With that in mind it is important to review case studies and investigations that others have performed so that lessons can be learned. The following case studies highlight some of the many studies that have been performed in the United States, as well as other countries.

2.2.1 Caltrans 14 Wall Study

The California Department of Transportation (Caltrans) undertook a survey and investigation of fourteen Mechanically Stabilized Embankments located across the state (Jackura et al. 1987). Since 1979 Caltrans has implemented the practice of installing non-stressed rods or coupons into the MSE backfill. These rods are removed and inspected at specific time intervals and the corrosion loss is measured and compared to the design assumptions. In 1985 Caltrans removed and inspected the sample coupons from a wall located in Mariposa County. There was severe pitting corrosion on the samples, which were only six years old. From these observations Caltrans decided to investigate the Mariposa wall site as well as thirteen other wall sites.

Although the Mariposa County wall experienced higher than expected corrosion, the other thirteen walls in the investigation were observed to have lower rates than the designed corrosion rates. The Mariposa wall was constructed with plain steel reinforcements that did not have a galvanized coating. However, three other walls also were constructed with plain steel reinforcements, but exhibited a more uniform corrosion and the loss rate was lower than the design rate assumed by Caltrans. The Mariposa wall experienced pitting corrosion and even when the pitted areas were excluded, the uniform loss was 167% of the Caltrans design values. It appeared that the galvanized reinforcements had good coverage and were performing in an acceptable manner. Caltrans' summary of other walls states that those with galvanized coatings did not have very many locations where the base steel had been exposed. It is interesting to note that the walls that were investigated were between three and fourteen years old. The current AASHTO design assumption (AASHTO 2007) is that the galvanized coating should last at least sixteen years. Therefore, it should not be a surprise that the younger walls did not have exposed base steel.

During the investigation Caltrans noted that there were differences in soil density between soils near the facing of the walls and soils found further from the wall facing. They noticed that there appeared to be a looser area within three feet of the facing while becoming increasingly dense as samples were retrieved farther from the facing. This is most likely due to constructions practices, including using lower compactive efforts near the first three feet from the facing. This practice proved to create an environment that was conducive to corrosion because of aeration differentials near the face and further back into the reinforce soil. More on this phenomenon can be found in Chapter 3. The other issue that was noticed at the Mariposa site was that the backfill consisted of a rocky fill and cohesive fines which differs from backfill that is commonly used. There is a similarity here to the next MSE wall that had a poor performance history located in South Africa.

2.2.2 South African Wall Study

At the Tweepad mining operation in South Africa, a series of Reinforced Earth Company walls were constructed in 1978 and 1979 at heights of up to 41 meters to support a gravity separation plant (Blight and Dane 1989). In 1980, there was a failure of a MSE wall at another mine in South Africa because of corrosion of the steel reinforcing strips in the backfill. As a result of this failure, the Tweepad mine became interested in the potential for corrosion of the soil reinforcements of their MSE walls. The investigation that resulted found that the metal strips had experienced corrosion in the form of severe pitting. Over the next several years the wall performance was monitored. The monitoring included strip tension measurements and outward deformation measurements. All of the walls at the complex showed deformation by rotation about the base without much translation. Eight years after construction the walls were removed and new MSE walls were constructed in their place.

The original construction practices, design methods and backfill materials were then evaluated because the shorter service life of eight years or 26% of the original thirty year design life posed a serious concern. During the design phase the electrochemical requirements of the backfill material were relaxed because the mine only needed a thirty year design as opposed to the seventy year design that typically accompanies the more stringent electrochemical requirements. The ancient beach sand containing high salt content was used for the backfill and it was compacted using sea water, which introduced more soluble salts, as well. Despite these corrosion concerns, galvanized strips with one millimeter of sacrificial steel were used and thought to be sufficient for the thirty year design life.

The Tweepad MSE wall investigation concluded that the most significant mechanism of corrosion was differential aeration. The cause of the differential aeration was the inclusion of clay lumps in the backfill. In areas where the clay lumps were in contact with the soil reinforcing strips pitting developed. As will be discussed in Chapter 3, differential aeration is caused by varying oxygen levels along the surface of the steel causing regions where differential oxygen levels occur to become anodic, which can result in localized pitting.

There were several lessons learned from this wall study. The reconstruction of these walls included a more controlled gradation of backfill with a limited amount of fines passing the 75 μm sieve in order to eliminate the prominence of clay lumps. Limits on electrochemical properties of the backfill, including the use of fresh water instead of sea water, were recommended for durability. With these modifications in the design and construction it is believed that the new walls will survive the design life required for mining operations.

2.2.3 Flamingo Wall Study

As discussed in the previous chapter, a series of three MSE walls at the intersection of I-515 and Flamingo Road in Las Vegas, Nevada is the starting point of the research presented in this report. In 2004, after approximately twenty years of service life, a contractor of Nevada Department of Transportation (NDOT) was excavating at the top of one of these MSE walls for the construction of new sound walls along I-515. During the process of the excavation some of the upper layer steel reinforcements were accidentally penetrated. The reinforcements that were visible appeared to be highly corroded, to the point that NDOT halted work on the sound walls and began a reinforcement corrosion investigation.

With the assistance and advisement of the Federal Highway Administration (FHWA), NDOT excavated several test pits at the top of the tallest wall. Soil and steel samples were collected and electrochemical tests were conducted on the soil. The results of the electrochemical tests, including soil resistivity, pH, sulfates and chlorides, showed that the backfill soils had significantly higher than recommended levels of sulfates. The soil resistivity was found to be significantly lower than the backfill that was approved during construction in 1985. The steel reinforcements in the MSE wall backfill were highly corroded with some of the steel bars having only pencil tip thick cross sections remaining. McMahon & Mann Consulting Engineers were asked to perform a more thorough and detailed investigation into the corrosion of these three walls. More test pits were advanced into the upper surfaces of these walls, samples were exhumed and measurements of the diameter of the remaining steel were taken at hundreds of locations. Backfill soils were also sampled and laboratory measurements including index testing and electrochemical testing was performed.

Some of the results of this investigation show that these three walls have experienced corrosion at a significantly higher rate than was anticipated during design. The wall was constructed using black or uncoated steel welded wire grids, which is not common in MSE wall construction. These steel reinforcements experienced significant pitting corrosion and it was noticed that the reinforcements located in the differential compaction zone directly behind the facing had developed pitting corrosion, likely due to differential aeration through the development of macro cells. The backfill soils consisted of generally silty gravels and as previously stated had high sulfate contents and low resistivity. The tallest wall, thirty-two feet at its highest point, was retrofitted with a tie-back wall because it was decided that the wall had experienced such a high amount of corrosion that it was no longer an effective retaining structure. The other two walls remain in place due to their lower heights. More on the analysis of this study and the measurements that were performed can be found in Chapter 4.

2.3 Soil Reinforcement Corrosion Surveys

There are three recent corrosion surveys detailed here. These surveys are aimed at collecting data on MSE walls across the United States and internationally. One of the underlying aims of each of these surveys is to obtain a better understanding of MSE wall behavior with respect to soil reinforcement corrosion. Because of the existence of these three surveys there was no necessity to develop another survey, since it would be similar to the three surveys detailed below.

2.3.1 AMSE Survey

In 2006, the Association of Metallically Stabilized Earth (AMSE) published a series of recommendations for AASHTO to consider with regards to corrosion of steel reinforcements in soil (AMSE 2006). Included in this publication was a request for AASHTO to consider reducing the corrosion rates used to calculate the amount of sacrificial steel required for the design of MSE walls. Using research and a survey of walls located in the United States and internationally, AMSE detailed its concerns regarding what it saw as the overly conservative current AASHTO guidelines. One of the main contentions raised in this 2006 paper is that, in general, MSE walls with galvanized steel reinforcements have behaved very well over the past thirty years (the typical design life is seventy-five years). There have been a few specific sites that have performed poorly, but there are issues that were found at each of these sites that do not fit within the normal trend of data across the United States.

A substantial number of investigations were cited by this 2006 paper that have been performed by state DOTs, other agencies, and member companies of AMSE. These results support the idea that many of the walls that have undergone investigations show corrosion rates equal to or less than the AASHTO guidelines require for design and construction.

When closely looking at the survey data, some very interesting observations can be made. The survey includes a catalogue of 780 MSE walls randomly sampled from the approximately 40,000 walls that have been constructed since the early 1970’s in the

United States. Almost half of the walls in the database are located in the western United States, and it should be noted that this database may only be representative of walls that have been constructed prior to 1990. From the survey data it can be seen that galvanized strip reinforcements were more commonly used throughout the United States, with the exception of the western states where galvanized welded wire mesh and barmat type reinforcements are more common.

As has been emphasized in previous discussions, the backfill material is the most significant factor in the corrosion issue. Of the thirty-eight states that provided 253 backfill records, there were 194, 133, and 130 records of measurements of resistivity, chlorides and sulfates, respectively. A large majority of the wall data that is presented shows soil resistivity data of greater than 10,000 ohm-cm, which is considered non-corrosive. As will be mentioned in Chapter 3, there is a relationship between resistivity and chlorides and sulfates (Figure 2, Elias 1990). However, it is commonly seen that resistivity is a useful overall predictor of corrosion resulting from salt content, including chlorides and sulfates.

Of the 780 walls in the AMSE catalog there are eleven wall locations that have been specified as poorly performing. This may seem like a small percentage, but high corrosion rates found in at least one of these wall locations (Nevada Flamingo Walls) was only discovered by accident during excavation of a soundwall footing above the MSE wall. It should also be noted that the resistivity of this backfill was assumed to be greater than 3,000 ohm-cm until electrochemical tests were performed subsequently. This is interesting because the survey results lead one to believe that only thirteen walls have soil resistivity at or below the 3,000 ohm-cm range. This may not be an entirely accurate representation of backfill characteristics, as will be discussed in Chapter 3, 4, and 6. It should be noted that there are very few walls in the United States that have been investigated with the use of physical measuring of cross sectional area loss.

Also, the pH appears to be within the ranges where, by definition, acidic or alkaline conditions do not exist (Figure 3). There are many references that suggest that pH is not a good measure of soil aggressiveness (Zhang 1996) as much of the time the pH falls between the accepted range of 5 to 10 for backfill soils. With the limitations in corrosion prediction using pH and a good portion of data falling within generally accepted values, minimal importance will be placed on pH data later in this paper.

One of the final issues that is presented by this survey is that of monitoring practices. From the survey results it appears that walls constructed between 1970 and 1980 are the ones most commonly monitored for metal loss. The southeastern U.S. far surpasses other regions in regards to conducting these monitoring practices. The metal loss monitoring practices are typically of the non-destructive type, such as polarization resistance and half-cell potential measurements.

2.3.2 NCHRP Survey

One of the more recent publications presenting data and information about corrosion practices and issues (for MSE walls, soil nail walls, and soil and rock anchors) in the United States is a National Cooperative Highway Research Program (NCHRP) project conducted for the Transportation Research Board and identified as NCHRP 24-28

(2007). An interim report presenting the findings of Phase 1 in a multiphase project included a ten question survey that was sent to all fifty States and Washington D.C. Thirty-two replies were received and several with additional comments to the initial survey. This survey was also sent to several jurisdictions in Canada, where seven of seven jurisdictions replied to the survey. One of the purposes of the survey was to identify the number and ages of metallic earth reinforcements located in each jurisdiction. Also addressed in the survey are questions regarding accelerated corrosion issues and corrosion monitoring practices. The survey concludes with questions about the potential willingness to share information and plans for wall demolition and reconstruction.

The specific data that has been collected in the survey portion of the project has been focused on inventory quantification. There are a couple of important conclusions that can be made from this survey. There are a large number of states that have dozens to hundreds of MSE walls. There are also a number of states that are willing to share their data with the NCHRP project.

The interim report also includes information regarding the specific findings of several studies that have been conducted on MSE walls. One of the more interesting summaries of information is located in a table discussing MSE wall locations that have detailed data. This table has been reproduced as Table 1. There are seven states that have been included in the table. There is a rating of backfill conditions that range from poor to good.

- Four of the seven states have backfill conditions that have a portion in the poor region of the range.
- Three of the four of these states, including California and Nevada, also have direct physical measurements that have been collected.
- None of the states with good ratings have any direct physical measurements.

This may be an important issue because as discussed with walls such as those found at the Flamingo site in Nevada, the outward appearance is not an indicator of a distressed condition. However, the Flamingo walls had undergone significant corrosion at higher rates than were anticipated.

2.3.3 Oregon Department of Transportation

In a research study, published in May 2008, performed by Oregon State University for Oregon Department of Transportation a survey of states and their MSE wall practices was presented (Raeburn et al. 2008). The questions of the survey revolve around the goal of obtaining information regarding the practices of other states with respect to materials used in MSE walls and corrosion issues. Altogether it consists of nine questions that focused on metallic soil reinforcement use, poor performance, and corrosion. There are not many quantitative results from this survey, which suggests that state DOTs do not really feel there is a problem with their MSE walls with respect to corrosion. The supporting evidence is that five of the eight responding DOTs said that they have not taken any measurements of corrosion rates on their walls. As has been seen in other MSE walls, without investigations into corrosion rates, either through thoughtful investigation (Caltrans Mariposa wall) or by accident (NDOT Flamingo walls) one cannot be sure that corrosion issues do not exist without proper monitoring of existing MSE walls. If state DOTs do not make observations to compare measured corrosion rates to the rates used in the design process, they could be in a similar situation to Nevada DOT who did not know of corrosion issues until accidental discovery. This can be a very costly method of corrosion monitoring.

One other conclusion that can be drawn is the significance of the use of steel reinforcements in the construction of MSE walls. Of the seven respondents, four states reported that 80% to 100% of their MSE walls were constructed with steel reinforcements in the past five years, including Nevada reporting 100%. This is significant because it has only recently been an accepted option for DOTs to construct MSE walls with materials other than steel, such as geosynthetic reinforcement which have different corrosion resistance characteristics. Five of seven respondents report that at least 80% of their entire MSE wall inventory consists of walls constructed with metallic soil reinforcements.

2.4 Soil Reinforcement Corrosion Recommendations and Practices

The information in this section is meant to provide a historical background for the development of today's corrosion standards and specifications. While this section reports on the limitations on soil properties, these values can vary greatly depending on which test method is used to measure these properties. Although AASHTO has specified certain test methods, many state DOTs have also selected their own test methods. Each of the test methods will not necessarily produce similar results. More discussion of these test methods and their potential for variation is included in Chapter 3. The information below is only meant to provide a context for how the specifications have evolved over time. These historical changes are related to laboratory corrosion testing and observation of existing structures and their performance. While some of the changes in the specifications over time can be linked to testing method and observation, some of the changes appear to be more related to the fact that corrosion in MSE wall inclusions is still not as well understood as engineers would like since the cost of destructive testing and mitigation is high, a conservative solution of providing sacrificial steel has become the preferred design approach.

2.4.1 Early Years

Many of the corrosion recommendations from the early years of steel reinforcement use were the result of field tests and observations from studies such as the NBS forty-five year tests and the French laboratory studies, as well as observations from buried pipe groups. The concept of an addition of sacrificial thickness, which was pioneered by the Reinforced Earth Company, was based on an assumed metal loss rate for the design life of the structure. In 1978 the French Ministry of Transport had electrochemical backfill requirements of a pH range of 5-10, a minimum soil resistivity of 1,000 ohm-cm, a maximum chloride content of 200 parts per million (ppm) and a maximum sulfate content of 1,000 ppm (Blight and Dane 1989).

Prior to this there are very limited quantifiable requirements that have been recommended. As will be seen below it was not until the 1990s that the American Association of State Highway and Transportation Officials (AASHTO) included any requirements into their Standard Specifications for Highway Bridges. It is interesting to note that a majority of the walls included in the AMSE survey (discussed earlier) were constructed prior to the incorporation of corrosion loss rates in AASHTO design. This is not to say that designers did not include sacrificial steel in their wall designs. However, these walls may have been constructed with more aggressive backfill or less sacrificial steel, or both.

2.4.2 FHWA

For more than twenty years the Federal Highway Administration (FHWA) has published guides and resources to assist engineers in the design of MSE walls. There are two publications that will be the focus of this historical background (Elias 1990). The first, FHWA-RD-89-86, published in 1990, presented a very thorough set of guidelines and theory of corrosion of steel and geosynthetic reinforcements buried in soil. This publication included the French and German study data, NBS Circular 579 corrosion concepts, as well as data and standards collected from other agencies, both internationally and in the United States.

One of the main objectives of the 1990 FHWA publication is to provide background theory and information regarding the current understanding with respect to corrosion issues specifically for MSE walls. Soil backfill electrochemical topics are discussed and there is a significant detailing of the tests and their strengths and weaknesses at measuring the important electrochemical properties. The corrosion mechanisms that result because of soil aggressiveness are also summarized. With the development of the background thoughts and current considerations FHWA presented corrosion rates that it felt were properly conservative to address both pitting and galvanic corrosion. More on these two types of corrosion are included in Chapter 3. It appears that AASHTO drew their first corrosion recommendations from this document.

In 2000, FHWA published a follow-up report that also addressed corrosion of MSE soil reinforcements, and both metal and geosynthetic reinforcements were included again (Elias 2000). This document is very similar to the previous version with respect to metal corrosion issues. Some of the modifications made in this version are more readable; however, some important information regarding test methods was excluded.

Both of these publications prove to be very valuable reading for metal corrosion background information. Much of the reasoning behind the current thoughts on corrosion rates and metal loss design considerations is presented in these publications. The strong point of the 1990 publication is that it discusses a variety of test methods that can be used to evaluate the electrochemical characteristics for backfill materials. However, the current practice tends to incorporate the test methods discussed in the 2000 version, making the 1990 version useful only as a historical context of some past practices.

2.4.3 AASHTO

The literature review of the AASHTO Standard Specifications for Highway Bridges began with the review of the eleventh edition (AASHTO 1973) because NDOT constructed its first MSE wall in Lovelock, Nevada in 1974. With a thorough review of the eleventh edition it was found that there was no reference to the design of MSE walls (or Reinforced Earth walls, as they were known at the time) in the Division 1 section and there was also no mention of corrosion or backfill properties in the Division 2, or construction section of the edition. It is worth noting that there is very little information about any retaining walls in this edition. This holds true through the fourteenth edition (AASHTO 1989).

In 1992 the fifteenth edition was released. With its release was a watershed of design specifications and requirements for retaining walls in general. More specifically, it is the first time that MSE type walls are mentioned. This is likely the result of the above mentioned 1990 FHWA publication (Elias 1990). Included in Division 1 design section are design life requirements of seventy-five years and 100 years for permanent and critical structures, respectively. The concept of sacrificial thickness was also included and standard corrosion loss rates were specified. In Division 2, the construction division, gradation and electrochemical limits are placed on the backfill materials used in construction of MSE walls. Although, it is important to note that there are no specifications regarding which test procedures should be used to verify the electrochemical limits of the backfill (AASHTO 1992). The AASHTO electrochemical specifications are presented in Table 2 in a timeline fashion including the NDOT specifications to show their relationship over time.

The sixteenth edition was presented in 1996. In this edition the specifications for standard corrosion rates and electrochemical properties were modified, but again did not specify the test procedures to use to verify these properties (AASHTO 1996). There were some other changes in the corrosion issues related the MSE walls in the seventeenth edition (AASHTO 2002). The electrochemical requirements were moved into the design section (Division 1), but more importantly the test procedures were specified as AASHTO testing procedures. Along with this inclusion there is more discussion with respect to corrosion including the following statement. “These sacrificial thicknesses account for the potential pitting mechanisms and much of the uncertainty due to data scatter, and are considered to be maximum anticipated losses for soils which are defined as nonaggressive” (AASHTO 2002 pg.152). Up until this edition, the epoxy coating of steel was included as an option. However, there is a note that states that there is no sufficient data to support the practice of epoxy coating the soil reinforcements. The most recent AASHTO publication is the LRFD Bridge Design Specifications (AASHTO 2007). Other than the significant change from ASD design methods to LRFD design methods the information regarding electrochemical testing and accountability of corrosion through the addition of sacrificial steel remains the same as the 17th edition.

2.4.4 Nevada Department of Transportation

A review of the Nevada Department of Transportation Standard Specifications for Road and Bridge Construction (also referred to as the Silver Book) gives a historical context for corrosion specifications in Nevada. Using the 1968 Specifications as a starting point there were no references to corrosion of MSE walls or MSE walls in general until the 1986 Silver Book. Between the 1976 and 1986 versions of the Standard Specifications there was at least one set of interim provisions. However, these memorandums were not readily available. The only records for these electrochemical corrosion requirements for backfill soils were found on the laboratory testing records of backfills that the contractor submitted for acceptance testing prior to use. One example of this can be found in the MSE backfill test data for Contract 1918. This set of electrochemical specifications is included in Table 2. More on the approval practices is discussed later in this section.

In the 1986 Silver Book there are several limiting characteristics of soils to be used in MSE wall backfill. These are presented in tabular form in a timeline relationship with AASHTO specifications (Table 2). The pH has an acceptable range between five and ten while the measured soil resistivity had a minimum limit of 3,000 ohm-cm. Both the chlorides and sulfates were bounded by maximum allowable values of 200 and 1,000 parts per million (ppm), respectively. These values compare well with the French Ministry of Transport limitations except for the resistivity minimum value which was increased from 1,000 to 3,000 ohm-cm. As discussed earlier, it was not until the 15th edition in 1992 that AASHTO published electrochemical specifications for MSE backfill.

The next Silver Book that was published was in 1996. In this publication, as with the 1996 AASHTO specifications the electrochemical specifications were modified to what they are today. Both the pH and resistivity remained the same when compared to the 1986 Silver Book. The permitted salt content was reduced for chlorides from 200 to 100 ppm while the sulfates were reduced significantly from 1,000 to 200 ppm. Similar guidelines have been presented by FHWA in their 1990 Task Force 27 recommendations, 1990 corrosion guidelines (Elias 1990) and the 2000 FHWA corrosion guidelines (Elias 2000). These electrochemical specifications are also the same as those found in both the current NDOT Silver Book (2001) and AASHTO specifications (2007 LRFD with 2008 Interim).

With the exception of one or two recent walls, the practice of acceptance testing prior to backfill use by the contractor has been the main method for measuring the electrochemical properties. The contractor will submit samples, NDOT personnel will test the soils and either approve the source or deny the specific source until further testing proves the source is acceptable. This will occur prior to the construction of the MSE walls. It should be noted that a review of the approved and rejected sources shows that in many instances a single source will provide backfill soil samples that are within and outside the specifications, but once the source provides material passes, that source is approved. There are a variety of questionable issues that are present in this practice. These will be discussed in detail in Chapter 4 and 6.

As previously mentioned, there are a handful of walls that have a different set of methods for the acceptance of MSE backfill. Several recent NDOT wall construction specifications (production testing) have required (e.g., I-15 North Design Built Project in Las Vegas) that the contractor stockpile a certain amount of potential backfill material on the jobsite prior to construction of the MSE walls. The acceptance test samples are taken directly from these jobsite stockpiles and then tested by NDOT personnel. If the material is accepted then it is used for construction. However, if the jobsite stockpile is rejected that stockpile cannot be used as MSE backfill. This provides a more controlled atmosphere where soils that are to be used are more representatively sampled and tested.

2.4.5 Local States

The literature review in this research included the review of the current MSE backfill corrosion practices for other western states surrounding Nevada including California, Oregon, Utah and Arizona. Western states have been the focus of this section because they are most likely to represent similar challenges with aggressive arid desert soils that are found in Nevada. Table 3 presents the electrochemical specifications for each of these states. When found, a test method is also specified for each electrochemical backfill property. This is a critical piece of information because of the wide variety of test methods used and the range of values that can be produced by each test for the same test sample. In many instances the chloride and salt content tests are not evaluated if the resistivity is a minimum of 5,000 ohm-cm. This is due to the thought that these soils will not likely have large salt contents and the soils can be classified as mildly corrosive (FHWA 2000). Specifications for states that have not been included in this table can be found on the FHWA state specification information clearinghouse (http://fhwapap04.fhwa.dot.gov/nhswp/index.jsp).

Along with the importance of soil aggressiveness is the type of steel reinforcement used. All of the states included in Table 3 have specified that the steel inclusions should be galvanized per ASTM A-123. This is an aim to insure that the corrosion rate will be similar or less than that of the specified rate loss of metals provided in the current AASHTO design guidelines.

Chapter Three
Corrosion Background

3.1 Corrosion of Buried Steel

Corrosion of metal reinforcements in soil is a natural phenomenon where the metal attempts to return to its fundamental state. Typical metals, like galvanized steel reinforcements, corrode back to a more stable state of salts and oxides when left alone in soil. It is important to have an understanding of the mechanisms of corrosion of steel in soil. With this understanding potentially problematic situations can be reduced or avoided altogether. Corrosion mechanisms require an environment that is conducive to electron transfer. One way to deal with this issue effectively is to control the environment in such a way that electron transfer is made more difficult. It has been found that there are certain variables that can estimate the ability of the soil to hinder electron transfer. These variables include soil resistivity, soluble salt content and pH. With limits placed on these variables of backfill soils, corrosion rates can be estimated and taken into account by way of additional steel and/or galvanization for a non-structural sacrificial coating. The following sections discuss the corrosion mechanisms, predictive measures and expected corrosion rates.

3.1.1 Corrosion Mechanisms

The corrosion mechanism for steel and other ferrous metals is typically electrochemical in nature. Electron flow from the anodic portion of the metal surface to cathodic portion is dependent on the existence of an electrolyte surrounding the metal inclusion in the soil. This exchange can typically occur at two levels, commonly referred to as micro and macro cells. While there are a variety of forms of these two types of corrosion, only two will be discussed here because of their significant effect on the corrosion experienced by MSE walls in typical backfill environments. These two corrosion types include pitting corrosion and galvanic corrosion for micro and macro cell corrosion mechanisms, respectively.

In the case of micro cells or localized corrosion, pitting corrosion is most commonly discussed in the literature, based on observations during site investigations and laboratory testing. Pitting corrosion occurs in areas on the metal surface where there are micro irregularities. These surface irregularities can create an electrolytic cell where an anode and cathode are developed. The area required for an anode is typically very small while the cathodic region is significantly larger. This cell can create a very large electrical potential that allows electron flow at an increased rate compared to the surrounding surface area. With this higher rate of exchange of metal ions from the anode to the cathode, significant metal loss can result. These areas are commonly referred to as pitted regions on the surface of the metal. These pitted areas are described as deep or shallow, where the deep regions produce deep cavities of metal loss over a small area and the shallow pits affect larger areas but are not as deep (ASM International 1987). Differential pH levels are created within the pit that, with an effective electrolyte, will allow the metal ions to leave the surface on the anode side of the metal and then be deposited on the cathode side of the electrolytic cell. Soil saturation plays an important role in the creation of an electrolyte. Without proper levels of saturation an effective electrolyte is not developed. In the French laboratory tests in the 1970s it was found that saturation ranges of 30% to 50% produced the highest potential for corrosion to occur (Darbin et al. 1988). In the case of the Flamingo walls the saturation levels were estimated as ranging from 25% to 40% (Fishman 2005). Many MSE walls will likely have an optimal amount of saturation because of the use of water to compact the soil, which is locked into place. There are seasonal variations in moisture content that can make electrolytic cells stronger during certain times of the year. The process of pitting corrosion is a self-sustaining process that can result in significant metal loss compared to situations where uniform metal loss occurs (ASM International 1987). Several studies, such as the NBS forty-five year experiment, have shown that pitting can occur at three to five times the rate of uniform metal loss (Romanoff 1989). While there is always a possibility that metals buried in soils can experience pitting corrosion, there are methods to reduce the likelihood or severity. The most common methods used in MSE wall structures include the use of mildly to non- corrosive soils and the use of galvanized coatings on the reinforcements (Scully 1990). The addition of this coating allows for more uniform corrosion by preventing pitting regions to form (Porter 1994). There is a plethora of evidence that shows that with the proper use of these two methods, namely prevention and protection, the probability of pitting corrosion is low (AMSE 2006).

The second type of corrosion that can be expected in steel reinforcements found in MSE wall backfill is typically called galvanic corrosion. In order to have galvanic corrosion, there are three main requirements that must be met. First, the metals must have different surface electric potentials. Second, they must also be coupled with the same electrolytic region. Third, there needs to be a common path where electron flow can occur. Geometry for this flow is critical. Areas where there are few corners and obstructions are more likely to have galvanic cells (ASM International 1987). In the case of metallic soil reinforcements, the geometry is very susceptible to galvanic corrosion. This is due to the fact that the reinforcements are typically welded wire mesh, barmat grids and long ribbed strips.

Two possible situations where galvanic corrosion can occur include backfill differences in aeration and moisture content. Aeration differentials across a metal inclusion, especially significant differentials over a short distance, can create an environment where an electron exchange can occur rapidly. An example of this was seen in the clay lumps in a gravelly soil as seen in the South African MSE wall study that was described in Chapter 2 (Blight and Dane 1989). There are also situations that occur in the typical construction of an MSE wall that can develop into a differential aeration. It is common to specify different compaction techniques directly behind the facing (typically in the first three feet) of an MSE wall (Elias 2000). In many investigations, including Flamingo, it was found that the level of compaction was lower in this region than at a farther distance from the wall. Although in the case of Flamingo the backfill was coarser pea gravel material near the face to aide the compaction process, which can also result in differential aeration due to grain size distribution changes. The primary reason for this practice is to avoid placement of heavy compaction equipment near the MSE wall facing which can result in larger lateral earth pressures and significant movements of the facing units. The practice of varied compaction techniques results in the development of two zones of different relative compaction. It has been found that the interface of these two zones is likely to develop an aeration differential resulting in higher corrosion rates than were found farther from the wall facing. The second possibility of galvanic corrosion in MSE walls is found where there are differences in levels of saturation along the metal reinforcement. There are several instances where this can occur. One of the most common includes infiltration of water at the boundaries of the MSE wall structure, such as water from roadway runoff at the top of the wall or leaking storm drain pipe works located within the reinforced MSE backfill.

While the potential for micro and macro cell corrosion may appear dire there is a natural phenomenon that can lessen the rates of these corrosion losses over time. In the case of steel reinforcements in mildly corrosive soils, a surface film around the metal can be developed to protect the metal from continual loss at the original rate. This creation of a surface film is referred to as passivity. Passivity is developed by a buildup or saturation of metal oxides which will create a layer of protection along the metal inclusion's surface. In characterized corrosion rates used for design the idea of passivity is included in the estimation of the loss rate of the galvanized coating. However, it should be noted that pitting corrosion is significantly more difficult to passivate than galvanic corrosion (ASM International 1987). In backfills that are considered moderately to highly corrosive it may be difficult to develop passivity for either case and corrosion rate loss assumptions become insufficient estimators of corrosion loss. Passivity is a very important assumption in the corrosion rate estimations that have occurred historically.

3.1.2 Corrosive Measures of Backfill

Although the mechanism for corrosion exists in all backfills, the rate at which this mechanism occurs is a function of the aggressiveness of the soil. There are several characteristics that define the aggressiveness of the backfill soil. These include the existence of bacteria and soluble salt content including chlorides, sulfates, and bicarbonates. Typically, by designating a limit of organic material in MSE backfill, the bacterial aggressiveness can be avoided. The soluble salt content is measured and controlled by several methods. It is common to limit the chloride and sulfate contents directly through backfill acceptance tests. Also very acidic and very alkaline soils (low and high pH, respectively) have been identified as having high salt contents. A majority of soils available for use as backfill material are within the moderate range with respect to pH (a pH range of five to ten). However, even with a neutral pH a soil can have a high soluble salt content. It has been found that a good predictor of soluble salt content is the measure of a soil's resistivity. The aggressiveness of soils has been classified by several agencies based on measured resistivity. Table 4 has a one of the more common ratings scale recommended by FHWA (Elias 2000). Because it is not reasonable to identify each type of salt contained in MSE backfill, many agencies will limit the chlorides and sulfates along with limits on pH and soil resistivity to account for other salts.

3.1.3 Estimated Corrosion Rates

As discussed earlier, corrosion is a natural and expected phenomenon. The rate at which corrosion or metal loss occurs is what engineers are concerned with in design, whether it is for pipelines, buried utilities, steel piling foundations, or MSE wall structures. As has been previously discussed there have been a variety of investigations into the rate at which this happens. Starting with the forty-five year NBS study a power equation was developed (Equation 2.1). From this power equation and further studies a simpler bilinear model was developed which included a higher rate of corrosion earlier in the service life of the buried steel and then an attenuation of metal loss over time due to passivity of the soil surrounding each steel inclusion. It has been found that the passivity of the surrounding soils typically does not occur for more aggressive soils because the electron transfer does not reach a level of exchange close to equilibrium within the typical design life of the structure.

It is common for agencies to use the bilinear loss model in specifying the metal loss that can be expected over the service life of an MSE wall. However, the bilinear loss model changes depending on the metals used in the inclusions. A review of the AMSE white paper survey shows that a majority of MSE walls are constructed using galvanized steel reinforcements (AMSE 2006). The loss model used for galvanized reinforcements has three phases in the AASHTO specifications (Figure 4). The first phase is the initial loss rate of the galvanized coating of 15μm/side/year. This rate is continued for the initial two years of service life. At that point the soil is assumed to be passivated and the rate is reduced 4μm/side/year until the galvanized coating is entirely lost. Many manufacturers provide a minimum of an 86μm galvanized coating for the steel reinforcements. Based on this minimum coating it is assumed that for a uniform metal loss the galvanized coating will be removed after sixteen years of service life (30μm in the first two years and 56μm over the next fourteen years). The third phase is the metal loss of the exposed bare steel. In less aggressive soils typically used in MSE backfill the soil surrounding the steel inclusion is passivated and a loss rate of 12μm/side/year is expected. This rate is assumed constant until the end of the design life of the structure. For a seventy-five year design life the steel reinforcements will be designed with a cross section to meet a specified tensile capacity and then a sacrificial thickness is added to the structural cross section. Typical sacrificial thicknesses that are added include an increase of 708μm/side of steel on top of the above mentioned 86μm/side of galvanized coating.

There are a few MSE walls, including the Flamingo wall in Las Vegas, which have been designed and constructed with steel reinforcements that have not been galvanized. Although the loss model is bilinear the rates are different than for galvanized reinforcements. This is due to the fact that the galvanized coating protects the steel and provides the means for a more uniform surface corrosion. Without the galvanized coating the initial rate of metal loss is significantly higher (Figure 5 and 6). However, with less aggressive soils that are typically used in MSE backfill the soil surrounding the steel inclusion is passivated and the rate attenuates to a similar rate seen in bare steel after the galvanized coating is corroded. As was stated earlier, it is not common to have soil reinforcements that are not galvanized because of the distinct benefits of reduced corrosion rates and increased potential for uniform corrosion.

It is important to keep in mind that a corrosion rate is based on the actual soil properties. For example, when the AASHTO metal loss rate is used in MSE wall design it is assumed that the backfill meets the soil electrochemical properties also specified by

AASHTO. When deviations from those soil properties are allowed there needs to be some compensation for the corrosion or metal loss rate that is to be expected. There are numerous recent studies that show a large number of MSE walls across the United States which have performed well and have experienced low levels of corrosion. These walls also typically meet the electrochemical guidelines specified by AASHTO. However, there are walls, such as the Flamingo MSE wall in Las Vegas, Nevada, that have performed poorly. During the 2004 investigation, the Flamingo walls were also found to have very corrosive backfill even though the original backfill approval showed electrochemical measurements were within the AASHTO specifications. Table 2 includes the NDOT and AASHTO historical electrochemical requirements for MSE backfill. Only small adjustments were made to the metal loss rates, but the electrochemical requirements have, in almost all cases, become more stringent. While the electrochemical limits appear to be equivalent, the methods used to measure these values are obtained by different means, particularly with respect to soil resistivity.

3.2 Summary of Electrochemical Testing Methods

In order to ensure that aggressive soils are not used in MSE wall backfill specifications have been identified by AASHTO to meet the corrosion models used in calculating the amount of sacrificial steel needed for a specific design life. As was seen in the discussion of the history of the specifications, it was not until recently that AASHTO included test methods in its specifications (AASHTO 2002). However, test methods were suggested in several FHWA guidelines in publications dating back to as early as 1990 (Elias 1990). These earlier suggested test methods revolve around ASTM tests, but in later FHWA manuals AASHTO test methods are suggested with the note that ASTM soluble salt content test methods are more accurate and reproducible (Elias 2000). Detailed below are the test methods for four typical electrochemical tests with special attention to the Nevada and AASHTO test methods.

3.2.1 Soil Resistivity

One of the best methods to evaluate the corrosive nature of soil is to measure its resistivity (Figures 7 and 8). There are a variety of standard methods that are used depending on the regulating body in charge of quality assurance. The three primary test methods discussed here include the NDOT Nevada T235B Standard Test Method for the Determination of Minimum Resistivity of Soil, the AASHTO T-288 Standard Method of Test for Determining Minimum Laboratory Soil Resistivity, and the ASTM G 57 Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method. Although there are many soil resistivity test methods in use across the United States these three tests will be the focus because they have been used in previously published documents directly related with the corrosion studies for NDOT at the Flamingo walls. Soil resistivity is a measure of the effective pathway of ion exchange in soil. It is primarily used as an estimate of the soluble salt content, or ions and cations that are dissolved in a saturated soil matrix. This indirect measurement of salt content is believed to be one of the better measures used to predict the corrosive nature of soils.

Current practice for NDOT projects, which dates back to at least 1980, and as specified in the Standard Specifications for Road and Bridge Construction (Silver Book), requires the planned MSE backfill soil resistivity to be measured using the NDOT Nevada T235B test method. This test procedure is based on the 1978 California Test 424 developed by Caltrans. The minimum limits in the specifications are 3,000 ohm-cm with limitations on the soluble salt content or 5,000 ohm-cm without limitations on the soluble salt content. The test is performed on backfill material that passes the #4 (4.75mm) sieve and this soil is soaked in distilled water for a minimum of twenty-four hours. After the soaking period, the distilled water is decanted into another container where the conductivity of the decanted water is then measured using a probe and the decanted water. Resistivity, the inverse of conductivity, is then reported to the nearest whole number. If there are discrepancies between multiple labs' test results NDOT specifies that the AASHTO T-288 Test method be used as the referee test method. There are no precision or bias statements for this method.

The procedure for the AASHTO T-288 soil resistivity test method is substantially different from the Nevada test method. In the AASHTO test method a device called a soil box is used to measure the resistivity. Soils that are tested are limited to material passing the #10 (2.00mm) sieve. Distilled water is added to the dry sample and allowed to hydrate for a minimum of twelve hours. The hydrated soil sample is then placed in the soil box and the resistivity is measured using a two electrode soil box. The soil is removed and more distilled water is added and mixed then the soil is returned to the soil box and the resistivity is measured again. This process is repeated until the resistivity has reached a minimum value. With this method of measurement the soil resistivity is directly measured at various saturation levels in order to find the worst case situation. It should be noted that the technician performing this test must make sure that the minimum value is actually reached. Some NDOT personnel have noticed that there is a possibility of a false minimum value with a slight rise in resistivity before dropping again. This is especially true for soil samples that are close to the specification limits and less important when the minimum resistivity of the sample is much greater or has been found to be lower than the specifications allow. There are no precision or bias statements for this test method.

The last resistivity test method to be discussed, ASTM G 57, is performed in a similar manner to the AASHTO T-288 soil resistivity test. The major differences are found in the type of soil box used and the saturation of the soil sample when the measurements are performed. The ASTM method specifies the use of a four electrode soil box. This soil box may be more precise in its measurements than the two probe soil box because of polarization effects. However, this test does not strictly specify that a minimum resistivity should be attained. There are some recommendations regarding saturation and compaction among others, but addressing those factors is up to the testing agency. There are precision and bias statements for this test method.

The first two test methods are most relevant to NDOT specifications at this time. As will be seen in the following section, the NDOT soil resistivity test method tends to over-predict the soil resistivity when compared to the AASHTO test method. A correlation between these two methods has been developed using data sets of soil tests where NDOT personnel have performed both tests side-by-side. During an interview with NDOT Materials personnel it was revealed that the Nevada test method produces different results on the same soil if the sample is allowed to soak for longer than the allotted twenty-four hour period (Blake 2009). It was suggested that future testing be done to compare the AASHTO resistivity test results to the Nevada samples where the Nevada samples are allowed to soak for longer time periods. This would provide the soaked samples time for the salts to go into solution. The ASTM test procedure is presented here because there are test results for field corrosion investigations that have been measured using the four electrode soil box and it is important for the reader to understand that there are potentially significant differences between all three test methods for the same soil.

3.2.1.1 Soil Resistivity Correlation

Through review of historical data it was realized that there were a number of data sets where series of individual samples had been tested using both the Nevada and AASHTO test procedures. There were several situations where these data sets were created. The first was a series of samples that were tested in a referee testing situation during a contract dispute between NDOT and a contractor. The second included a number of samples that the NDOT materials laboratory used to compare time differences between the two methods. Most recently NDOT materials personnel have been conducting the soil resistivity tests side by side on MSE backfill samples to compare the results. The Nevada test method is still the approved method in the NDOT Silver Book, but a case can be made as to the significant difference between the two methods and the results they provide from the same soil samples.

A collection of 114 tests was found through the historical review and data collection of MSE walls in Nevada. In all of these cases the each sample was tested using the Nevada T235B soil resistivity test method as well as the AASHTO T-288 test method. Not all of the tests were performed on MSE wall backfill. Nine of the 114 tests or less than 8% of the tests were conducted on backfills to be used in other applications such as general backfill and base material supporting pavement. However, all were conducted on granular backfill samples. A plot of the data is presented in Figure 9 on a log-log scale. A regression analysis shows that there is a correlation between the two tests and that the NDOT test results can be converted to estimated AASHTO test results and that the Nevada test method consistently over-predicted the resistivity of soil samples. The following correlation equation has been developed as,

$$y = 0.859x^{0.963} \tag{3.1}$$

where x represents the measured Nevada soil resistivity and y represents the estimated AASHTO soil resistivity. This can be useful in evaluating historical MSE wall data for NDOT walls. The percent difference between the two test methods is, on average, 31% different. With the consistent over-prediction of resistivity, which is not conservative, using the Nevada test method it will be likely that there are a number of walls that have resistivity values lower that what was desired and that the soils used may be more corrosive than anticipated. The plot in Figure 9 include the minimum value lines demarking passing and failing test results for both test methods in order to distinguish the differences between the test results. It is interesting to note that of the 114 test comparisons ninety-one, or approximately 80%, fall below the minimum resistivity requirement set by AASHTO while only forty-three of the 114 tests, or approximately 38%, fall below the 3,000 ohm-cm minimum Nevada requirement. Further evaluation shows that forty-eight of the 114 tests, or 42%, are higher than the required minimum resistivity when using the Nevada test method, but would not be considered acceptable when using the AASHTO test method (data points located in the lower right hand side of the graphed quadrants). This evaluation supports the concept that the Nevada T235B test method, as it is performed, produces results that are less conservative than the AASTHO T-288 soil resistivity test method.

While this is a useful comparison it should be kept in mind that more test results will assist in developing a stronger relationship. The R-squared term of 0.925 (base 10 log units) would suggest a strong correlation while the plotted residuals show that the data may not have enough randomness to provide a strong correlation (Figure 10). The t-distribution test produces a value of thirty-seven, thus proving that the correlation coefficient, R-squared, to be statistically significant. The F-distribution test results have a value of 1,386 which proves that the null hypothesis tested should be rejected and the correlation is significant statistically. This correlation can be seen as one of many tools that can be used in evaluating the likelihood of increased corrosion rates in existing MSE walls throughout Nevada. For example, a sample that would be accepted with a measured resistivity of 3,000 ohm-cm using the Nevada test method can be converted to an equivalent value of 1,916 ohm-cm with the AASHTO test method. However, using the original value of 3,000 ohm-cm, which would be categorized as "moderately corrosive" based on Table 4, will now be considered "corrosive" with the AASHTO converted value of 1916 ohm-cm.

3.2.2 Soluble Salts

While the measure of soil resistivity is an indirect measure of the soluble salt content of a backfill soil there are two soluble salts that are evaluated during a typical acceptance process. These two salts are chlorides and sulfates. In the following sections the Nevada and AASHTO tests methods for determining these two salt contents are discussed. While these test methods do measure a portion of the salts in the backfill soils they do not necessarily give a complete measurement of either salt. It is suggested by Elias that one of the only ways to make a complete measurement of these salts and any others is to perform chromatography tests such as those used in ASTM D-4327-88 (Elias 2000). While neither of these tests includes chromatography, they do give a basic approximation of the chloride and sulfate contents and, in the case of the AASHTO test, are widely used across the United States to quantify the existence of these two salts.

3.2.2.1 Chloride Content

It has long been recognized that chlorides are more aggressive and cause corrosion at a higher level that sulfates (Figure 11). In order to measure the chloride content in backfill soils, there are several tests that can be performed. Many of the major testing agencies, such as ASTM, AWWA, EPA, Caltrans, Nevada, and AASHTO have testing procedures to measure the chloride level. AASHTO does have a recommended testing procedure, T-291, which is specified in their current specifications for MSE wall backfill practices. NDOT also has a test procedure used for the same measurements. Both of these methods are discussed below. Although the other methods from different testing agencies were incorporated in other MSE wall corrosion research in Nevada they will not be addressed here since these tests are not as common in MSE backfill approval for state DOTs. However, it is important to bear in mind that different test methods will produce different results, and some of those may be significantly different. Also, because of this variation in results by method it may not be acceptable to combine and average results from different test methods.

The AASHTO T-291 Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil contains two options for measuring the chloride content of MSE backfill material. The first method is a Mohr titration method and the second method that is suggested in this test procedure is a pH/mV meter analysis. There are no precision or bias statements for this test method.

Until 2004 NDOT performed an unnumbered test method for both the chlorides and sulfates. In the earlier test for chloride content, a backfill soil sample (portion passing the No. 4 sieve) is saturated and agitated and then soaked for twenty-four hours. The liquid solution is decanted off and a measured amount of potassium chromate is added to the decanted solution. This solution is titrated to its endpoint with silver nitrate. The chloride content is estimated by multiplying the milliliters of silver nitrate added to the solution. There are no precision or bias statements for this test method.

After a memorandum in January 2004, the materials testing laboratory at NDOT was informed that all chloride and sulfate tests were to conform to AASHTO T-291 Method A and AASHTO T-290 Method B, respectively. Although the directive was given in 2004 it is likely that the change in test methods was not made until sometime in 2005 (Blake 2009). There were no correlation evaluations performed to identify any differences between the NDOT and AASHTO test methods. Hence, it is difficult to make any strong statements regarding the differences between the two methods.

3.2.2.2 Sulfate Content

As has been discussed previously, sulfates are less aggressive than chlorides. However, from the review of the Flamingo backfill test data sulfates were present in significant levels compared to chlorides. Therefore, it is important to discuss the methods of quantification for this soluble salt. As with the chloride discussion, there are a large number of test methods that can be used to measure the sulfate content in backfill soil. These include methods from ASTM, AWWA, EPA, Caltrans, Nevada, and AASHTO. Although these methods have been used in backfill testing for the Flamingo MSE wall investigation these tests do differ from each other. Only the NDOT and AASHTO test methods will be discussed in this section, primarily because, in the instance of the NDOT test, it was used extensively for approving historical backfills and the AASHTO test method is the currently recommended method for state DOTs to incorporate in their approval processes. It cannot be overstated that these different tests produce differing results and should not be combined without great care and correlation.

In the same memorandum dated January 2004, NDOT materials testing procedures were to change from the Nevada test method to the AASHTO T-290 Method B for measuring the sulfate content in MSE backfill soils. Prior to this time the method for measuring the sulfate content was a very obscure method with a high likelihood for differences to result between technicians. The original preparation of the sample is the same as the chloride test method. In the test for sulfate content a backfill soil sample is saturated and agitated and then soaked for twenty-four hours. The liquid solution is decanted off and filtered then a measured amount of methyl orange indicator is added to the decanted solution. This solution is titrated to its end with sulfuric acid then barium chloride is stirred in and the sample is allowed to stand for a few minutes. If there is any cloudiness in the solution's appearance, a few drops of hydrochloric acid are added. If the cloudiness clears then it is a result of carbonates in the solution and if the cloudy appearance continues then it is due to the existence of sulfates. To measure the sulfate content the solution is poured into a hand-marked graduated tube and the tube is placed between the technician and a fifty watt light bulb. At the point where the cross over the light bulb becomes blurry the measure on the graduated tube is read as the sulfate content. There are no precision or bias statements for this test method.

This test has been replaced by the AASHTO T-290 Method B (Turbidimetric Method) test to measure sulfates. The AASHTO test method consists of a sieved sample passing the No. 10 sieve that is saturated in distilled water and centrifuged. The sample is filtered, if required, and then glycerin and sodium chloride are added. At this point the filtered solution is mixed with barium chloride and placed in a photometer and measured.

This number is then converted to a sulfate ion content based on the amount of other constituents that were added to the solution during the test. There are no precision or bias statements for this test method.

There are some obvious issues with these two test methods. There is a significant amount of error that is introduced in the measurements provided by the Nevada test method. In the AASHTO T-290 scope discussion it is stated that Method A (gravimetric method) is a better method of characterizing the sulfate content but that Method B produces results more quickly. It should be noted that these two tests are not necessarily good measures of the actual sulfate content. FHWA does state that the chromatography analysis, as used in ASTM D-4327-88, is still the best method to evaluate the actual sulfate content in a MSE backfill soil.

3.2.3 Soil pH

While the pH level of soils used in backfill is routinely measured, there is a broad range within which they can fit and still be approved. As has been detailed earlier, that range has a lower bound of 5 and an upper bound of 10. As was seen in the AMSE survey (Figure 3) there are very few soils outside of this range (with a significant majority falling between seven to nine) that have been used as backfill in MSE wall applications (AMSE 2006). The pH values from approved backfill in Nevada MSE walls support this conclusion as well. The distribution of pH measurements, from 92 approved samples show that the values range between 7 and 9, with seventy-one samples having a pH of 8 (Figure 12). A plot of resistivity versus corrosion for NDOT approved backfill shows that there is no clear relationship between the two measurements (Figure 13). With the scatter depicted in this figure it can be observed that the two measures are independent and cannot be used to evaluate each other. Early studies of corrosion of galvanized steel culverts and review of other studies have attempted to relate the soil pH to a rate of corrosion, but have found that it is not a good predictor of potential corrosion rates (Zhang 1996). However, the importance of pH, especially within the limited range of MSE wall specifications has been judged to be of lesser importance than soil resistivity and salt content. With this in mind the test methods for determining pH will not be discussed here. However, for the reader's information, NDOT typically uses the Nevada T-238A test method while FHWA recommends the AASHTO T-289 test method for determining soil pH (Elias 2000).

Chapter Four
Nevada Case Studies

Since 2004, there have been two MSE wall corrosion investigations conducted in Nevada by NDOT personnel and others. The first was conducted in 2004 at the I-515/Flamingo Road intersection in Las Vegas, Nevada. The second was conducted in 2008 by NDOT personnel and the author, at the I-15/Cheyenne Avenue intersection, also in Las Vegas, Nevada. At each of these intersections the MSE walls were discovered to have suffered significant corrosion. These discoveries were based on chance due to excavation or removal for other projects. During field visits to these sites it was noticed that each of these wall locations did not seem to show any outward signs of distress that would cause the observer any cause for concern. Although these wall corrosion investigations have been referred to in other chapters they will be further detailed here. From the study of the conclusions drawn from each of these investigations better predictions can be made with respect to other MSE walls in Nevada, and more specifically in southern Nevada.

4.1 Flamingo Walls

The MSE walls constructed at the intersection of I-515 and Flamingo Road are a series of three Hilfiker Retaining Walls. These walls were designed in 1984 and constructed in 1985 to exposed facing heights of up to thirty-two feet. The walls were constructed using welded wire fabric (WWF) grids varying in diameter with depth. The steel used in the WWF was not galvanized and left bare. It was known at the time that this practice was not common but that with sufficient sacrificial steel the soil reinforcements would behave in a satisfactory manner.

As mentioned previously, the MSE wall corrosion investigation at the Flamingo walls was initiated because of observations of highly corroded soil reinforcements. The soil reinforcements were found by accident in January 2004 while a contractor was excavating sound wall footings at the top of the tallest of the three walls along I-515. During the excavation process some of the soil reinforcing steel was uncovered and on-site personnel observed that some of the steel had corroded to such a state that there was very little cross section remaining. NDOT advanced three test pits to depths of three feet below existing grade at the top of this wall (Wall #1) to make further observations. At that point NDOT contacted FHWA with the thought that certain engineering decisions needed to be made with respect to the stability of the walls at this site. FHWA and NDOT made observations of soil reinforcements in six test pits advanced to depths of six feet below the existing grade behind the facing at the top of the tallest wall, Wall #1 of 3. At that point it was decided that this MSE wall did not have the structural capacity to perform as designed for the seventy-five year design life required.

The next steps were two fold. NDOT hired a consulting company to design a retaining wall that would provide a seventy-five year design life while not requiring the removal of the existing MSE wall, which would disturb the traffic flow of I-515 significantly. The resulting design suggested mitigation by constructing a cast-in-place tie-back wall in front of the existing MSE wall. At the same time NDOT hired McMahon and Mann Consulting Engineers, P.C. (MMCE) to assess the condition of the three walls at the intersection and provide estimates of the remaining service life of the soil reinforcements for the remaining two walls that had not been mitigated (Fishman 2005).

4.1.1 Field Investigation

There were several methods used to sample reinforced soils from behind the MSE walls. Soil samples were collected through borings behind the walls during the design of the sound walls and for the design of the cast-in-place tie-back wall constructed as a mitigation effort for the first wall. NDOT and FHWA excavated nine test pits at the top of Wall #1 and MMCE advanced a total of four test pits to depths of five feet below the top surfaces of Walls #2 and #3. Backfill samples were also retrieved from behind lower sections of the MSE walls by advancing test pits through the facing of the walls. FHWA personnel collected samples of soil from nine locations while MMCE collected soil samples from seven other test pit locations advanced in the MSE wall facing.

During the field investigation activities MMCE also instrumented the three walls with monitoring stations where measurements of half-cell potential and corrosion rates could be observed indirectly. These monitoring stations are connected to forty-five existing soil reinforcements and to thirty-six “dummy” coupons (non-stressed metallic inclusions) including bare steel and galvanized steel rods. Measurements at these monitoring stations were conducted using a polarization resistance (PR) monitor, and these stations can be used for future measurements as well.

4.1.1.1 Information Collected

Testing the soil and steel reinforcements collected from these borings and test pits provided insights into the properties of the soil and corrosion rates experienced over the approximately twenty years of service for the Flamingo MSE walls. Index testing and electrochemical testing of the retrieved soil samples included sieve analysis, moisture content, organics content, pH, resistivity, chloride content, and sulfate content. Four testing laboratories were used during the testing process. However, there was no duplication of tests performed by multiple labs. One other issue with the testing by four different testing laboratories is that different test procedures were used by each of the laboratories (Table 5). This produces results that are difficult to compare, as discussed in Chapter 3, and should not necessarily be averaged to identify an average overall condition. Statistical analysis of the electrochemical test results between laboratories is presented in later in this chapter. These test results are also compared with the original data from 1985 for soils that had been approved for use in the backfill during construction of the three walls.

While soil measurements are useful to identify the corrosive nature of the environment the most useful measurements include the WWF diameter measurements from steel samples removed from behind the three walls. Twenty-nine reinforcement samples were retrieved from Wall #1 and eighteen were collected from Walls #2 and #3. These samples each contained several wires, both longitudinal and transverse in orientation. MMCE personnel measured the diameters at approximately 2,800 locations on these thirty-seven samples. The locations that were measured were brushed with a wire brush and pliers prior to measurements and in cases where the cross section loss did not appear uniform three measurements were performed at 120° apart and averaged to use as one approximate diameter.

4.1.1.2 Testing Results

The testing of the soils and steel samples produced several results that pointed to a corrosive environment and steel that corroded at a higher than anticipated rate. In general, the soil near the facing of the wall had a different gradation and compaction than the soil found further away from the wall facing. The backfill soils consisted of poorly graded gravels, silty gravels, and poorly graded sands to silty sands. From the electrochemical testing results the soils in the backfill were found to have a resistivity of 1,000 ohm-cm and a sulfate content of 660ppm. The median pH was 9 and the chloride content was typically below 50ppm. These characteristics identify the soils used in MSE wall backfill as being corrosive (Table 4). These characteristics are far different from the accepted limits in the wall specifications that were approved for use and were detailed on the 1985 materials test reports. This is a cause for concern, and in later sections we intended to undertake statistical comparisons to deal with this inconsistency.

The investigators for MMCE found that the steel sample measurements, both direct diameter measurements and PR monitor measurements, appear to be similar with respect to estimated corrosion rate. The calculations performed for the direct measurement diameters work towards an assumed uniform corrosion rate measurement that can be compared with the PR monitor measurements, which produces a uniform corrosion rate measurement. The uniform measurement produced by direct measurements is calculated by “the integration of the measured diameters divided by the total length of the wires included in the sample” (Fishman 2005, pg. 7). From this it was found that the corrosion rates are estimated to range from 5.2 μm/year to 29 μm/year with a mean corrosion rate of 14 μm/year. This can be compared to the PR results of 0.75 μm/year to 76 μm/year with an average of 11.8 μm/year and 8.9 μm/year for the months of March and August, respectively. The change in average corrosion rates between March and August is likely due to seasonal variations in saturation in the backfill. Several of the steel samples were examined using x-ray spectroscopy techniques. These metallurgical analyses found that there were no anomalies in the steel that would make it perform better or worse than the typical steel used in soil reinforcement.

4.1.1.3 Flamingo Field Investigation Conclusions

The field investigation performed by MMCE produced several interesting observations. They characterized backfill as very corrosive. The steel samples were observed to have corroded at least two feet from the front facing of the walls, to distances of at least five feet from the facing (the limits of excavation), which indicates that macro cell corrosion occurred. The PR measurements provided similar results to those found from idealized uniform corrosion loss calculations of the directly measured diameters. This is an important comparison because no steel samples were collected at depths greater than five feet from the top surfaces of the walls. It can be assumed that the correlation between the direct measurements and PR measurements at shallow depths can be extrapolated to greater depths and one can assume that the steel located at these greater depths has experienced similar corrosion rates.

Calculations of remaining service life found that the grids used in the soil reinforcement had 38% of their original capacity and that Walls #2 and #3 had locations where the steel been stressed between $0.69f_y$ and $0.78f_y$, where the design capacity was likely to be $0.48f_y$. It was also observed that the ratio of the average maximum corrosion loss and the idealized uniform corrosion loss ranged between 3 and 5. This is also an observation that had been made at other sites by other investigators. There are two methods for calculating anticipated corrosion. The MMCE report used a ratio of years of anticipated corrosion (the point where measured data “fits” the expected loss model) divided by the service life of the reinforcement (b/a ratio in Figure 14). Using this method, the walls were found to have experienced 2.5 times the corrosion rate that is typically anticipated. This corrosion rate is from the stress calculations, and not from the PR and direct measurements. These observations were found to hold true for all three walls.

4.1.2 Further Analysis of Data Collected

The results of the investigation identify the MSE walls at the I-515/Flamingo Road intersection as highly corroded and in need of repair. However, review of the diameter loss measurements led the author to evaluate the corrosion loss using a slightly different method. It is this author’s opinion that more weight should be placed on the direct diameter measurements and the corrosion rates that can be obtained from these measurements. The methods of polarization resistance and half-cell potential measurements have been found very useful as non-destructive tests, but these measurements produce an estimated corrosion rate that assumes a uniform corrosion loss. Review of the diameter measurements shows that uniform loss did not occur, therefore this assumption is less than accurate. Even though the idealized loss rates calculated from the direct measurements show good comparisons to the PR measurements, there is an inconsistency between this comparison and the calculation of remaining service life based on the evaluation of stress capacities of the grids. This fact has also been observed by others (Chapter 2), especially in aggressive backfills, such as those found in the Flamingo backfill. More attention has been paid to the direct diameter measurements by the author with the goal of identifying a corrosion rate that can be used to extrapolate future corrosion behavior in the wall backfill for the two remaining walls.

4.1.2.1 Corrosion Rates from Direct Diameter Measurements

The diameter measurements performed by MMCE were taken and further analysis of corrosion rate, based on cross sectional area loss, was undertaken. These calculations were performed for all locations measured. However, special focus was placed on Walls #2 and #3 because they have not been mitigated at this time and the location of the samples collected were identified in a more precise way than those found in the Wall #1 data. Summary statistics of the each type of calculation have been included in Table 6. There are three ways that the diameter measurements were used to calculate corrosion loss, which, in turn, can be used to evaluate a loss model that is more appropriate for the Flamingo wall backfill. The first is to calculate an estimated corrosion radial loss rate to compare with values used during the original design process. It is also useful to calculate the ratio of actual corrosion to the anticipated design life corrosion rates. This ratio, defined as the corrosion severity ratio, represents a normalized rate of corrosion where the expected rate of corrosion is normalized to a value of one (d/c in Figure 14). Figure 15 shows the distribution of the corrosion severity ratio data. Any corrosion that occurs at a higher rate, as observed from diameter measurements, will have a ratio of observed to anticipated rates greater than one. As an example, there are 38 of the 275 diameter measurements that have experienced rates of corrosion at six to eight times higher than was anticipated. This graphic is a clear indication that there are higher than normally expected rates of corrosion occurring in the Flamingo MSE wall backfill. The third evaluation that can be conducted for insight into the corrosion behavior is to calculate an estimated corrosion life. This was undertaken by assuming the bilinear model suggested by FHWA, which identifies the corrosion history. From this the diameter loss can be approximated by pseudo-service life ages. As the age increases the diameter decreases, and an approximate overall pseudo-age of specimens can be compared to the actual age of the walls. In this case the actual age of the walls are twenty years, but the pseudo-age of the walls are much older. Descriptive statistical results from these three evaluations are included in Table 6.

An evaluation of the diameter measurements from Walls #2 and #3 shows that the distribution in not precisely normal (Figure 16). This observation requires careful evaluation of the descriptive statistics detailed in Table 6, where the mean and median values are not equivalent. The use of the mean or average value to approximate the corrosion rate that can be expected may not be appropriate in conservative designs. Therefore, a statistics based approach is attempted below. Some may feel that the median value is a more appropriate statistical parameter to use when the distribution is not clearly normally distributed. The confidence interval of 95% was also used so that the likely range of the mean can be estimated. The fourth statistical parameter that can be used for corrosion rate evaluation, which is commonly used in earthquake risk analysis, is the use of the 84th percentile. Using these four statistical parameters a corrosion model can be developed that can predict future wall behavior at Flamingo and can lead to a better characterization and will constrain the loss rates that will eventually lead to wall failure. The models that have been created are based on Equation 2.1, where the “k” values have been adjusted, using standard “n” values from other research, to “fit” the measured radial loss at the Flamingo walls with their twenty years of service life. Subsequently, one can extrapolate the loss rate over time.

A comparison of the results obtained by evaluating the different statistical results from the Flamingo diameter measurements is useful when identifying how the MSE wall reinforcing elements will behave over time under the current corrosive environment. Table 7 identifies the “k” values calculated from the area loss statistics found in Table 6. Based on research conducted by NBS (Romanoff 1989) and FHWA (Elias 1990), an “n” value of 0.80 is seen as a representative value for bare steel that did not have a galvanized coating (Elias 2000). These estimated loss models can then be compared to the bilinear loss model (Figure 5) and the power loss model (Equation 2.1) used by FHWA. Elias notes that the power equation that is suggested in the NBS study may not be representative of the more restrictive backfill used in MSE wall construction. This difference is seen when comparing these two predictive loss models in Figures 17 and 18 (grey linear and curvilinear lines). On the right side of the figure there is a designation between the two loss regions. The first loss region includes the loss of sacrificial thickness as per the FHWA 1990 bilinear loss model (Figure 5). Ideally this thickness will only be reached at the end of the design life of the structure. The upper region represents the loss of structural steel. Once the sacrificial steel is removed the structural section begins to experience loss. This is problematic because the structural section provides the tensile stability for the MSE wall.

It should be noted that this loss is calculated based on direct measurements, and is not based on a uniform loss assumption. By looking at the comparisons of the average loss measurement and the confidence interval of that calculation in Figures 17 and 18, it can be seen that these values predict a significantly more corrosive environment than the backfills expected by FHWA and are somewhat more corrosive than what would be predicted by the NBS model.

In order to compare the Flamingo measurement data further, an analysis using Caltrans 1984 design criteria was conducted. The Caltrans 1984 design criteria was discussed in a corrosion investigation report conducted by Caltrans in 1987, and discussed in Chapter 2 (Jackura et al. 1987). Caltrans developed this set of criteria after reviewing the NBS report data and identifying characteristics they felt were important predictors of behavior. In these criteria soils were identified in six different classes, starting with select granular backfill versus normal backfill. A review of the gradation curves presented in the Flamingo investigation identified the Flamingo backfill as "normal backfill", and not "select granular backfill". The three different backfill types in this classification include normal and alkaline, acidic, and corrosive categories. These three predictive curves for normal backfill are included in Figure 19. These curves are based on area loss due to corrosion. The measured Flamingo data point, with 95% confidence interval, for twenty years of life was included on the figure to identify where the measured loss is located on the range of backfills. The Flamingo measured loss data identifies the backfill to be less corrosive than the corrosive backfill identified by Caltrans. As seen in Table 8, Caltrans has identified corrosive backfill as having a minimum resistivity of less than 1,000 ohm-cm. The Flamingo backfill has a slightly higher soil resistivity average value of 1,000 ohm-cm (Fishman 2005). This relationship does not identify any differences that may exist as a result of the differences between soil resistivity test methods.

From the diameter measurements performed during the MMCE investigation predictive corrosion rates have been calculated. It is interesting to note that the PR monitoring measurements and the idealized uniform loss calculations discussed in a previous section showed that the corrosion rate was approximately 14 μm/year. To put this in perspective, the corrosion rate that would be expected from bare steel samples in backfill that is considered to be mildly corrosive should be about 15.3 μm/year according to the FHWA 1990 bilinear loss model for black steel (Figure 5). PR measurements relate well with uniform corrosion loss calculations according to Fishman (2005). But analysis with point-by-point direct measurement data is more appropriate for failure analysis as breakage can occur anywhere along the reinforcements. The estimated radial corrosion loss is, on average, 56 μm/year. If these walls are going to fail they will not fail because of an "idealized" corrosion rate based on uniform loss. They will fail because of localized corrosion creating cross sectional areas that do not meet the capacity required to carry the wall loading, either under static or seismic conditions.

4.1.2.2 Evaluation of Backfill

The backfill electrochemical test data, measured at the time of Flamingo construction in 1985, shows that the resistivity is suitably high and the salt contents that were measured were low enough to satisfy even today's requirements (Table 9). There are five backfill sources that were approved during the construction. Initially a total of nine samples were tested. Only the five shown in Table 9 were approved for use as MSE backfill. It is not clear which of the five approved backfills were used during the construction process, which means that all tests need to be included in the evaluation because it is a possibility that at least one, and possibly all approved sources were used at some point in the construction of the three MSE walls at the Flamingo intersection. The approved backfill test data from 1985 in Table 9 shows many of the samples would have been approved in 1985, with the exception of one test, if today's NDOT specifications (2007) were used.

As has been discussed in detail in Chapter 3, there is a significant difference between the AASHTO T-288 and Nevada T235B soil resistivity test methods. In order to evaluate that data in a meaningful way, based on the correlation between the test methods, the Nevada test values have been converted using Equation 3.1 to estimated AASHTO values. Table 9 also shows that only, two of the five samples would have been rejected if the converted AASHTO soil resistivity test results had been used. The historical timeline of the backfill specifications can be reviewed in Table 2. These observations do not provide clear proof that an evaluation using the 1985 approval data would be cause for concern with respect to corrosion.

While the 1985 data suggests that the MSE backfill is only moderately corrosive, the 2005 data suggests that the backfill is actually very corrosive. The response of the soil reinforcements supports this finding. The data collected during the MMCE investigation in 2005 is included in Table 10. The soil resistivity test method is also included in this table. With a brief glance through the data measured in 2005 it is obvious that the backfill is corrosive. The same evaluation that was conducted for the backfill approved for construction in 1985 can be repeated here. These comparisons are presented in Table 11. A review of the data that was measured for backfill samples retrieved in 2005 shows a large range of values. A variety of test procedures were used to evaluate the backfill properties. These test methods are identified in Table 5. The first three test results produced by NDOT (specified by MSE Fill as the backfill sample location) were produced using the Nevada soil resistivity test method. These three results have been converted to an estimated AASHTO soil resistivity using Equation 3.1 for better comparability with other AASHTO test results. It is unfortunate that there was no replication of tests between testing laboratories so that repeatability of these test procedures can be evaluated. It is also unfortunate that all of the four electrochemical tests were not conducted on all of the samples, although this is likely due to the fact that many of these samples were obtained by soil borings and a larger sample size would be required to conduct all of the testing. Of the nineteen soil resistivity tests conducted, six would have been approved (approximately 32% passing) for either specification time period (1986 and 2007). When evaluating the chloride contents there are only three of thirty tests which would have been rejected in 1985 and currently (90% passing). There is a stark contrast when an evaluation of the sulfates is performed. There are fourteen tests out of thirty that do not pass when using the 1986 criteria (approximately 53% passing), but only five of these are passing tests when the 2007 test specifications are used (approximately 17%). It is obvious from the evaluation of these recent backfill tests that the sulfate content and the low resistivity prove that this backfill can be classified as very corrosive. Of the twelve samples where all three electrochemical tests were performed there are only three, or 25%, that would be approved under today's standards. These basic evaluations are useful in identifying that the backfill used in construction may be different than the backfill that was approved for use during construction in 1985.

It is generally believed that the backfill, once placed behind the walls, does not undergo a significant change in electrochemical composition throughout its depth. In order to identify whether there is some significance in the differences between the originally approved backfill and the backfill that was sampled and tested in 2005 a statistical evaluation is required.

The Analysis of Variance (Anova) is a useful statistical tool that can be used to evaluate the correlation between datasets of backfill properties obtained in 1985 and

2005. The hypothesis tested here was whether there is any statistical difference in properties between the backfill that was approved in 1985 and the backfill sampled in 2005. In addition, the Anova analysis used the data from different laboratories as independent datasets. This is important because it can lead to an understanding of issues that could be affecting other walls in Nevada. In these statistical evaluations a sample size greater than one is required. Therefore, the sample tested at the Geotechnics laboratory was not included. A systematic removal of outliers was also performed in order to reduce their effects on the analyses.

It may be noted the proposed statistical evaluation can provide more insight into the similarities or differences between backfill approved in 1985 and the 2005 investigation, and results from laboratory results, indirectly evaluating the differences between test methods. SAS Macro FIXQL, developed by Dr. George Fernandez, was used to perform the Anova analyses with SAS 9.1 statistical software (Fernandez 2009). The program assumes equal treatment of variances using the general linear model (GLM). The first evaluation is that the data relationship has a probability greater than the F-statistic (Pr>F). If this value is less than 0.05 there is statistical significance between at least one of the datasets compared others. There are four datasets that were analyzed including the 1985 NDOT approved backfill, the 2005 NDOT tests, the 2005 Terracon – Sparks tests, and the 2005 Terracon – Las Vegas tests. The Least Significant Difference (LSD) Analysis was also used to identify statistical differences between datasets and to support other evaluation methods (Kuehl 2000). This analysis did not evaluate results for a Bonferroni analysis because this method is conservative and can, in some cases, mask the existence of statistical significance. Analysis was conducted for soil resistivity, chloride content, sulfate content, and pH. The statistical evaluation of each of these measures is discussed in greater detail in the author's thesis (Thornley 2009). A summary of the results of the analysis are discussed below.

4.1.2.2.1 Statistical Evaluation of Soil Resistivity Test Results

There are four distinct sets of data that have been included in this analysis. The original data collected from 1985 approved backfill testing reports is compared to the 2005 results of the three testing laboratories, namely NDOT, Terracon – Las Vegas, and Terracon – Sparks. A graphical presentation of the average values and ranges of each dataset is included in Figure 20. As mentioned previously, the data from Geotechnics was removed because it had only one sample test. One of the assumptions that is made in this evaluation is that the backfill that is tested is fairly uniform and similar. The original data was evaluated statistically first. The data that is evaluated has not been modified from the original test results, even in the cases where the Nevada test method was performed. In this instance the original approved backfill test data is statistically different when compared to all three laboratory datasets from the 2005 investigation. From this analysis it can be seen that the recent datasets also are statistically different from the originally approved test data. This may be a result of using different test methods or it may be that the approved backfill is dissimilar from the backfill sampled from behind the Flamingo walls.

An example of the type of analysis that was performed for the electrochemical properties of the backfill is included for the above case. The Anova evaluation gives a value of 0.0215 for the Pr>F statistic using the GLM procedure. This provides the initial insight that at least one set of data is statistically different from the others because the value obtained is less than 0.05. A P-value of 0.0923 supports the assumption that the values in the datasets are normally distributed. In order to identify which sample groups are statistically different the results in Table 12 can be reviewed. If the value in the table detailing the least squares means values is less than 0.05 there is statistical significance between the two compared datasets. In this instance the original approved backfill test data is statistically different when compared to all three laboratory datasets from the 2005 investigation. A further evaluation, using the LSD method (critical value = 2620) provides a graphical evaluation of the results (Figure 21). A review of the residuals, presented in Figure 22, shows that there are no outlying data points.

Although it is interesting to make a general review of the original datasets it is important to realize that these datasets should not really be compared directly because of the differences in test methods. The three test methods used for resistivity measurements (Nevada T235B, AASHTO T-288, and ASTM G57) are distinctly different methods of measurement. A correlation, presented in Chapter 3, between the AASHTO and Nevada soil resistivity test methods has been developed. In this case it is useful to repeat the above statistical analysis using direct AASHTO resistivity measurements along with converted AASHTO values. In order to accomplish this, the test data from 1984 was converted to AASHTO estimated data using Equation 3.1. There are also three NDOT test results from 2005 that were acquired using the Nevada test method. These have been converted as well. The test data from Terracon – Las Vegas has been excluded because it was obtained using the ASTM soil resistivity test procedures and no correlation has been developed between the AASHTO and ASTM soil resistivity test methods. This data is presented graphically in Figure 23, showing the average values and ranges of the datasets. The analysis of this data also showed that the originally approved backfill is significantly different from the backfill sampled in 2005.

There are two conclusions that can be drawn from these results. The first is that the statistical evaluation presents a case where it appears that the originally approved data are statistically different from the backfill that was sampled in 2005 from behind that MSE walls at the Flamingo intersection. The second is that there is a fair amount of variability in test data. As stated in Chapter 3, there have not been any precision or bias statements for the AASHTO T-288 soil resistivity test method. This questions the procedure of measuring statistical significance with a small dataset. Either way the backfill is evaluated, the backfill that has been used in the construction of the MSE walls at the Flamingo intersection are much lower resistivity than measured resistivity of the backfill samples tested in the approval process.

4.1.2.2.2 Statistical Evaluation of Chloride Content Test Results

A similar statistical method of analysis has been used for chloride. Again, there are four sets of data that can be evaluated in this case. The 1985 test data only provided two results from the five source samples. An Anova evaluation shows that there is statistical significance between at least one of the datasets when compared to others. However, the P-value was less than the value of 0.05 required in order to support the assumption of normality in the data. Figure 24 shows the measured data ranges and average values. Without looking at a larger collection of datasets, or datasets with similar test methods, it becomes a difficult task to identify why the results are varying depending on the statistical approach. With a P-value that does not support the normality assumption the statistical significance found between datasets should be used with care.

4.1.2.2.3 Statistical Evaluation of Sulfate Content Test Results

It may be noted from Table 11, the sulfate limits have been exceeded by most of the samples. The four datasets are presented graphically with their respective averages and ranges in Figure 25. The main issue is that there is a large amount of data that identifies higher sulfate values than is currently allowed. This higher content, in conjunction with low soil resistivity, supports the fact that the soil is more corrosive than those backfills preferred for use in MSE backfill.

4.1.2.2.4 Statistical Evaluation of pH Test Results

Even though pH has not been a focus of the evaluations for corrosive backfill because of the normal range where the data fall, thoroughness requires the statistical evaluation of the pH test data here as well. While the pH measurements are within the specified range of 5 to 10, as presented in Figure 26, there is statistical difference between at least one of the datasets compared to others. The statistical differences found may either be due to the different testing procedures or variation in soil samples. However, all of the test results meet the specifications for pH in MSE backfill.

4.1.2.3 Potential Effects on Wall Stability

While it is useful to compare the electrochemical test results and the diameter loss measurements, it is more useful to use this knowledge to evaluate the potential stability issues to the existing MSE walls at Flamingo. The results from the previous two analyses support the conclusion that the Flamingo walls have experienced high rates of corrosion because of corrosive backfill. NDOTs mitigating construction of a tie-back wall in front of the MSE Wall #1 also supports this conclusion. However, there are two remaining walls that have not been mitigated. With the development of predictive loss rates outlined in previous sections this evaluation, it is possible to address the stability concern.

The approach for the analysis of the two remaining MSE walls is based on the current practice for MSE wall design and analysis, as presented by AASHTO in the 4th edition of the Load and Resistance Factor Design (LRFD) bridge design specifications (AASHTO 2007). Using this approach with the design methodology currently used an analysis of the existing wall internal stability based on tensile strength of the soil reinforcements has been conducted for both of the remaining MSE walls at the Flamingo intersection. Both static and seismic evaluations were conducted. The two seismic cases that were used include the design motion at the surface of a_{max} = 0.15g identified by NDOT Bridge Division and also the input motion of a_{max} = 0.21g calculated from United

States Geological Survey (USGS). The calculations for the USGS input motion is included in Appendix A.

From these LRFD static and seismic analyses a capacity to demand ratio is calculated (C/D ratio), replacing the technique (Allowable Stress Design – ASD) of calculating the Factor of Safety, as has been common in the past. The load and resistance factors are included in each calculation instead of using a factor of safety, resulting in a need to have a C/D ratio greater than one for adequate design and analysis. Design characteristics that are similar in both walls are included in Table 13. The evaluated sections of Wall #2 and #3 have effective heights of 32 and 15.5 feet and have the geometries as shown in Figures 27 and 28, respectively. These geometries are the same as those evaluated by MMCE, however their evaluation was based on a factored tensile capacity of the wall reinforcements at one instant in time. Using the predictive loss curves developed in the earlier section, the wall behavior can be evaluated over time.

When using the LRFD method, a factor is placed on the yield stress of the steel. This effectively keeps the yield stress of the soil reinforcements within the linear-elastic region of the stress-strain behavior of the steel. When evaluating the life expectancy of these MSE walls the full yield strength of 70ksi is used for both static and seismic cases. It should be noted that the difference between static and seismic response may be smaller than one might expect given the lower design excitation level at Las Vegas. This is due to the fact that the yield strength is multiplied by a resistance factor for the seismic case of 0.85, while the static case uses a resistance factor of 0.65. The stability calculations assume that the reinforcements will fail along the edge of the backfill failure wedge as seen in Figure 29. Sample MSE wall internal stability calculations have been included in Appendix A.

Two estimated loss models are used for analysis. The first estimated loss model evaluates the results of corrosion if the soil reinforcements experience losses at the average power loss model (k = 103 in Equation 2.1) calculated from the diameter loss measurements. The second loss model evaluation identifies the wall behavior expectations if the 84th percentile loss rate is assumed (k = 180 in Equation 2.1). For structures that have importance and safety requirements, such as these retaining walls, this is an appropriately more conservative estimate of the expected behavior, especially because the full yield stress is included in this stability analysis. When evaluating other permanent structures it is common to evaluate the 84th percentile (average value plus one standard deviation) case when there is some uncertainty, in order to be conservative.

The tensile capacity of the soil reinforcements can be compared to the tensile load introduced by the backfill soil on the soil reinforcements. As a baseline case, the original steel cross sections are used to calculate initial internal stability of Wall #2 (Figure 30). A C/D ratio can be calculated by dividing the capacity of the reinforcement strength by the stress applied by the static and seismic loads. This baseline analysis has also been used to evaluate the initial internal stability of Wall #3. With the baseline case established, further analysis accounting for corrosion using the power loss models has been conducted. The results of stability calculations estimating the corrosion of the Flamingo walls are presented for three time periods in a snapshot fashion. These snapshots are in twenty-five year increments starting with a twenty-five year service life.

As the time progresses, calculated capacity becomes smaller, as a result of the corrosion losses. In the case of Wall #2 the average corrosion loss rate tensile values are presented in Figure 31. After seventy-five years there is very little to no capacity remaining in the reinforcements.

Using the average power loss model, the regions where a W7 WWF grid has been used there is an estimated corrosion loss of 3,260μm per side, equivalent to 0.128 inches per side. The original bar diameter is 0.298 inches, as specified by Hilfiker Retaining Walls. This results in roughly 0.042 inches of diameter remaining, or an equivalent cross sectional loss of 98%. This is an overall loss that includes sacrificial and structural sections.

When evaluating the 84th percentile predictive loss model the situation is significantly worse. In the case of Wall #2, which is constructed using three different WWF sizes, including W7, W9.5, and W12 with diameters of 0.298, 0.348, and 0.391 inches, respectively as specified by Hilfiker Retaining Walls (Figure 27), there is no remaining steel cross sectional area after about forty-five years of service life. However, it is more likely that Wall #2 would fail prior to complete loss of cross sectional area. Using the 84th percentile analysis with the larger two longitudinal bars it is apparent that the remaining cross sections of the W9.5 and W12 bars will be corroded to zero cross sectional area at approximately 55 years and 60 years of service life, respectively.

The LRFD procedure of stability analysis is based on the C/D ratio. In Figure 32 the C/D ratio values are presented in the same snapshot method for the static case, using the average power loss model. Figures 33 and 34 present the two possible seismic responses based on an input motion of 0.15g and 0.21g, respectively. In the case of Wall #3 the average corrosion power loss model tensile values are presented in Figure 35. In Figure 36 the C/D ratio values are presented in the same snapshot method for the static case. Figures 37 and 38 present the two possible seismic responses based on an input motion of 0.15g and 0.21g, respectively.

The 84th percentile tensile capacity values for Wall #2 are presented in Figure 39. These tensile values can also be evaluated in a clearer fashion by observing the C/D ratio values in the static and seismic cases. Wall #2 values are presented in Figures 40 through 42 for the static and two seismic cases, including the 0.15g and 0.21g input motions, respectively. The figures presenting the Wall #3 tensile capacity and C/D ratio values are located in Figures 43 through 46.

There are several potential failures that can occur as these two walls continue to corrode. The use of either of the loss models identifies that there is serious concern with the future stability of Flamingo MSE Walls #2 and #3, as is seen in Figures 32-34, 36-38, 40-42, and 44-46. The estimated number of service life years each wall can expect until a C/D ratio of 1 is reached is presented in Table 14 for both the average and 84th percentile power loss models.

In the case of Wall #2 there are three zones resulting from the use of three different diameter longitudinal steel bars were used varying with depth (Figure 31). Wall #3 has only one zone of reinforcements because it included the use of only W7 reinforcement grids. The bottom portion of each of these zones experiences higher horizontal stresses than the top of the zones. This will likely be the location of failure for each zone. There is another consideration for this failure to happen. The zone of corrosion should be at the edge of the active portion of the backfill and not in the active wedge directly behind the wall facing as shown in Figure 29. As one layer of grids looses tensile capacity the horizontal stresses will be shed to the grids above and below the failed grid resulting in higher stresses in the surrounding grids, which have experienced similar high rates of corrosion. This distribution of horizontal stresses will likely have a domino effect and the walls may not be effective as retaining structures. The evaluation of the failure progression is outside of the scope of this evaluation. However, a review of the case study of the South African Mine MSE walls, discussed in Chapter 2 will give some insight into wall behavior as MSE walls approach failure due to higher than anticipated rates of corrosion. The MSE walls in South Africa experienced rotation with little translation about their bases with time. As the measured deformations continued to increase, the owners decided that even a few centimeters of outward movement at the height of forty-one meters was too much and the walls were demolished and replaced.

4.2 Cheyenne Wall Study

In September 2008, two University of Nevada, Reno civil engineering graduate students, including the author, and a representative from NDOT travelled to Las Vegas, NV to visit an intersection where a portion of an MSE wall was being removed and replaced. The existing wall is a Reinforced Earth Company galvanized tie strip wall and the new wall was designed to be the same type of wall. The contractor had removed a portion of the existing MSE wall for realignment of the southbound on-ramp for I-15 from Cheyenne Avenue. The existing MSE wall was constructed in approximately 1998 to 1999 under Contract Number 2853. The new construction work is under Contract Number 3313. This work included removal of approximately 110 ft of existing MSE wall and replacement with a newly aligned 344 ft section of MSE wall. The new wall was designed to be connected by a slip joint to the remaining existing wall.

The contractor began removing the existing wall and informed NDOT that there appeared to be an unusual amount of corrosion on the soil reinforcing steel (Figure 47). Upon arriving at the site it was found that all of the planned removal had occurred. There were removed panels near the site and some steel strips placed in a pile with rebar scraps (Figure 48). Most of the strips had been cut from the facing units making it difficult to identify the location in the wall (vertical location), distance from facing unit, and which part of the strip was the top surface during service life. There was a stockpile of removed backfill material that was located near the wall.

Several measurements and observations were noted while evaluating the panels located onsite. It was difficult to know the orientation and location of each panel was located unless it was a top panel. The top panels had Styrofoam pieces near the top where the coping had been placed on top of them (Figure 49). Many of the other panels that were not top panels had obvious corrosion damage, suggesting that corrosion had occurred at the surface and at depths below the surface. It appeared that there was variability in corrosion locations. Some of the bolts and nuts used for connecting the strips to the panels showed significant corrosion, others showed more corrosion on the nut side, and others showed no corrosion at all. None were noticed to have any significant corrosion on the bolt head and with no corrosion on the nut.

A small survey of the panels at the site found that there was a random distribution of panel connections that were corroded. This randomness presented itself both on individual panels and from panel to panel. Two of the panel connection pieces that showed significant loss in section were measured along their edges to quantitatively evaluate the section loss. Three edges of the connection are exposed while the fourth is encased in concrete in the panel. One of these connection pieces had lost its entire thickness due to corrosion in one spot and a hole had developed to nearly a half inch in diameter (Figure 50). The other had lost significant section thickness along one edge. The thicknesses measured can be found in Tables 15 and 16. For reference, a section of a connection piece that had not experienced obvious corrosion was measured and it was found to be 0.192 inches thick. Using this comparison a percentage of remaining thickness was calculated and is in parentheses next to each measurement in Tables 15 and 16. Based on the assumption that the AASHTO loss model (Figure 4) is applicable, at ten years of service 0.038 inches should be lost from each side for a total loss of 0.076 inches. Approximately 60% of the original thickness should remain after seventy-five years of service life. However, as seen in Tables 15 and 16, there are already two measurements that have surpassed this value at only nine years of service while other measurements are approaching the 60% thickness value.

4.2.1 Sampling and Measuring of Soil Reinforcements

After a short site reconnaissance a number of reinforcing strips were chosen for further analysis. Strips varying from no apparent corrosion to strips with significant loss of steel were selected for measuring and evaluation. These strips were transported to the University of Nevada, Reno and were given identifications, photographed, cleaned and measured. The cleaning process included using a stiff wire brush and brushing surfaces that were to be measured. In order to clean the surfaces more thoroughly, a pickling process, which removes the corroded galvanized coating, would need to be used. This process was not readily available, so the surfaces were brushed. Therefore, the thickness measurements may be less conservative with respect to section loss because of over prediction of remaining section thickness. However, many of the cross sectional areas did not have any remaining galvanized coating on the surface.

In all, there were twenty samples that were identified as facing connecting strips because of the hole located at one end of the strip, and seven strips that had been cut at both ends. The length of soil reinforcing strips that were collected for measurements totaled to more than forty-six feet. With the intention to identify the cross sectional loss or remaining capacity for the strips, several measurements were performed. Unlike a majority of the Flamingo wall bar measurements, the strip cross sectional area estimation required several measurements at each cross section. The width of each of the selected corroded sections was measured. Then the thickness of both edges at the identified cross section was measured. If there appeared to be any uneven thickness distribution across the section a central measurement was taken. The width measurement multiplied by the average thickness measurement was calculated to represent the cross sectional area. A total of ninety-nine cross sections were estimated from 304 measurements producing an average of more than two cross sectional estimations per foot of sampled soil reinforcing strip. The measurements were obtained by using a caliper with a sensitivity of ±0.0005 inches.

During the site investigation, four soil samples were selected for evaluation. NDOT personnel performed soil resistivity, pH, soluble sulfates, and chlorides testing for the samples. NDOT procedures regarding test methods were used for the selected electrochemical tests. However, in order to compare to AASHTO test results, the soil resistivity tests were performed using both the NDOT and AASHTO test methods. This allowed for further comparison to between NDOT and AASHTO test method results. Three samples that were collected were from randomly chosen locations. One sample was taken from the stockpiled backfill material located onsite and two samples were taken at the northern end of the removed section of the MSE wall. One of these sample locations was a few feet from the previous face of the wall while the other was taken from the same location perpendicular to the wall face but approximately twelve feet into the backfill (horizontally). The fourth sample was taken from backfill soil surrounding a strip that had not been removed and appeared quite rusty.

4.2.2 Cheyenne Analysis – Soil Reinforcements

After the cross sectional areas were estimated, an evaluation of the section loss was performed. The original design calculations were not available for this evaluation.

With the absence of the original calculations, including calculated sacrificial thickness, other similar designs from the late 1990s were used instead. It is a fairly common practice to use 86μm galvanized coating and a sacrificial steel thickness of 885μm for a total thickness of 971μm per side to be achieved. From a quality control standpoint, it is likely that the original thickness on the reinforcing strips was thicker in order to ensure that there was a minimum galvanized coating thickness of 86μm. Measurements of the strip thicknesses that appeared to be in good condition show that this is a fair assumption. Therefore, the original cross sectional area used in calculations is based on only the minimum required sacrificial thickness. This is a conservative assumption with respect to average cross sectional area loss estimation, but would be less conservative for estimating corrosion rate loss. It is interesting to note that the design calculations that were reviewed in order to estimate the original cross sectional area showed that designers only included thickness loss and did not include width loss in their corrosion loss calculations with respect to sacrificial thickness. This is a practice that was noticed in many of the collected contract design calculations for Reinforced Earth Company walls.

4.2.2.1 Estimated Corrosion Rate

Following the same practice that was used in the Flamingo wall analysis, estimated corrosion rates have been calculated for the removed Cheyenne MSE wall. The backfill soils used in the construction of the MSE walls at Cheyenne have produced higher than anticipated levels of corrosion. The measured loss of cross sectional area presented in Table 17 has been used to evaluate the corrosion rate that occurred with respect to the expected or AASHTO design corrosion rate. The distribution of corrosion rates that were calculated from loss measurements is shown graphically in Figure 51. From this it can be seen that the distribution is not strictly normal, and this is corroborated by comparing the cross sectional mean and median values in Table 17. The mean value suggests that the corrosion rate is more than six times the expected corrosion rate for a nine year old wall, using the corrosion severity ratio (d/c in Figure 14). Based on current understanding and accepted practice, the Equation 2.1 power loss model has been used to estimate future corrosion based on the measurements that were obtained after nine years of service life. The use of an “n” value of 0.65 is common to both Darbin and Elias and is present in their recent publications (Darbin et al. 1988, Elias 2000). From the use of this “n” value and knowing the measured loss statistics, a number of “k” parameters can be estimated and used in evaluation of expected corrosion loss models (Table 18). From these Figure 52 is developed. Also included in Figure 52 are the NBS galvanized steel model and the AASHTO bilinear model. The sacrificial thickness loss and the structural steel thickness loss regions have been identified so that it can be appreciated that with a higher than expected corrosion rate, the structural cross sectional area will be lost, which results in a direct loss of tensile capacity as the service life increases. Therefore, as with the Flamingo wall evaluation future stability calculations should be evaluated using the mean and 84th percentile values.

An evaluation of the measured cross sectional area loss can also be evaluated using the Caltrans 1984 design criteria for backfill, as was done with the Flamingo data. The percentage of area loss with 95% confidence interval is plotted in Figure 53. The sieve analyses performed for the backfill testing identify the Cheyenne backfill as being in the normal range and not “select granular fill”, as specified by Caltrans. The area loss is higher than would be expected when the electrochemical measurements in Table 19 are compared to the backfill classification specified by Caltrans in Table 8. It would be expected that the area loss would plot to the right of the corrosive backfill curve in Figure 53. Therefore, the electrochemical test results underestimate the corrosiveness of the backfill with respect to the loss measurements. There is significant difference between the low corrosion normal backfill curve and the loss data, which supports the general concept that there is a corrosion problem at this wall location.

4.2.3 Cheyenne Analysis – Backfill Soils

As stated previously, four soil samples were retrieved directly from the MSE wall backfill at I-15 and Cheyenne Avenue. These samples were tested and then compared to the results from the original backfill approved for use in the original construction in 1998. The results from the laboratory testing are reproduced in Table 19. Highlighted cells identify results that do not pass the current electrochemical specifications. A graphical representation of the ranges of the measured electrochemical properties is included in Figures 54 through 58. The 2008 testing occurred at an opportune time because a systematic evaluation of the Nevada and AASHTO soil resistivity test methods was being pursued, as discussed in Chapter 3. This allowed NDOT to evaluate the backfill using both the Nevada T235B and AASHTO T-288 soil resistivity test methods. Figures 54 and 55 identify the stark contrast between the results produced by these two test methods. While there are test results that fall below the minimum resistivity using the Nevada test method, the average and range of data that do not pass increases significantly when the AASHTO soil resistivity test method is used.

Below is a discussion of the statistical analysis of each of the electrochemical properties measured during the approval of backfill in 1998 and backfill evaluation of backfill in 2008. The statistical analysis uses the same Anova analysis that was used in the Flamingo evaluation. The only difference is that there are only two datasets instead of four and the overall number of samples is smaller.

4.2.3.1 Statistical Evaluation of Soil Resistivity Test Results

Two data sets were used in the statistical evaluation of the soil resistivity test results. The first evaluation used the Nevada test method results from both datasets and compared their statistical similarity. The initial analysis shows that there are no statistical differences between the two datasets. The NDOT test procedure does not have any precision or bias statements, where the datasets can be evaluated further.

The second, more interesting evaluation incorporates the use of the AASHTO measurements in the 2008 dataset and AASHTO correlated results using Equation 3.1. The converted data range is identified in Figure 54. Results from the Anova analysis on these two datasets suggested that there is statistical difference between the two data sets. This is to be expected when measured corrosion rates are higher than the predicted corrosion rates.

4.2.3.2 Statistical Evaluation of Chloride Content Test Results

The chloride content measured for each dataset shows that both datasets have values that are lower than the maximum values allowed for backfill approval. However the datasets are evaluated to identify significant changes from approved backfill in 1998 compared to the samples in 2008. However, a post-priori power analysis suggests that there is not enough data to support or reject this result.

4.2.3.3 Statistical Evaluation of Sulfate Content Test Results

While the statistical analysis of the chloride contents has found that there is no significance between the two datasets, this is not true for the sulfate content. There is a significant difference between the two datasets. It is interesting to note that this was also the case for the Flamingo analysis. This may be a result of backfill differences, but it may also be related to the fact that NDOT changed testing procedures, as discussed in Chapter 3.

4.2.3.4 Statistical Evaluation of pH Test Results

The pH values measured in 1998 and 2008 were evaluated to identify if there is any statistical significance. The datasets, when evaluated using the Anova statistical analysis, suggest that there is statistical significance. With possible statistical differences in pH and statistical differences in sulfate contents and resistivity results, the statistical analysis suggests that there is some difference between the backfill that was approved in 1998 and the backfill that was sampled in 2008.

4.2.4 Further Evaluation of Cheyenne Walls

The portion of the MSE wall at Cheyenne that was evaluated no longer exists. However, there are several other MSE walls that were constructed in 1998 that still exist. The wall design details and shop drawings could not be located during this research. Without the design details, further evaluation was not produced using the estimated corrosion rates from power loss models , as has been done with the Flamingo MSE walls. However, knowing that there is an issue with increased corrosion rates at this wall, it would be prudent to perform an analysis of the existing walls. It is apparent that the soils approved for construction were, for the most part, within the required specifications, but the backfill sampled and tested from behind the Cheyenne wall are not within the limits used in practice today. Other walls at this site may be in the same condition and should be evaluated with this possibility in mind.

4.3 Concluding Remarks Regarding Both Case Studies

There are several conclusions that can be drawn from the review of both of these case studies. The conclusions revolve around the results from the backfill testing analysis and the direct measurements from corroded metal reinforcements retrieved from the MSE walls. The data suggests that the soils are more corrosive than originally believed during the approval process. The resulting corrosive environment has produced significantly higher rates of corrosion than was anticipated during the design of these MSE walls. This directly affects the stability of these walls. It should be noted that current thoughts in the MSE wall industry suggest that walls with galvanized coatings will perform adequately under the current conservative AASHTO guidelines, and that walls such as the Flamingo wall performed poorly because it did not incorporate galvanized coatings. This may be generally true, but as seen at Cheyenne, there is specific and great potential that southern Nevada walls may be at risk to higher than anticipated corrosion rates.

The backfill statistical analysis suggests that the approved backfill may not be the same material used during construction. There are several conclusions that can be drawn from the statistical evaluation of the backfills from both walls. The first is that the backfill that was approved during construction is not statistically similar to the backfill used during construction. There are some possibilities to explain this difference. One option is that the approved backfill is not the same as the backfill used in construction. The other is that the testing that was done did not characterize the material effectively. This leads to the second conclusion. The historical testing methods that were used in Nevada to characterize the backfill are not adequate in identifying the corrosive nature of the backfill used in MSE walls. The sulfate and resistivity test results in both wall cases identify significant differences between the approved backfill and the backfill that was used. This is either because the test methods are not effective or because the materials were different. This is an important issue because there are more wall locations in Nevada that could be experiencing higher rates of corrosion because corrosive backfills may have been used in construction. The following two chapters will identify the number of walls that NDOT has and also present a methodical and systematic way to evaluate them.

Chapter Five
Nevada MSE Wall Database Development

It is obvious from Chapter 4 that there are at least two wall locations in Nevada that have experienced high rates of corrosion. In order to identify other walls that may have the potential to be affected by high corrosion rates an analysis of the walls that have been constructed is required. A first step in this undertaking is the development of a database of existing MSE walls. Such a database of Nevada walls has been developed containing a list of forty-one locations where at least one wall has been constructed. Of these forty-one locations it is believed that forty of them still have MSE walls, as only one wall location in Washoe County was removed from the inventory because it was removed during a lane widening project. The following sections detail the development of the database and other information that was collected in order to aide in the selection of walls that should be investigated for potentially higher rates of corrosion than were anticipated during design.

5.1 MSE Wall Data Collection

The information required to develop a comprehensive database was found through a multifaceted approach, including review of contract bid items, construction records, as-built drawings, materials testing data, and others. The first task was to develop a listing of all of the wall locations in Nevada within NDOT's sphere of influence. Starting from contract bid records dating back to 1986, NDOT personnel created a spreadsheet of structures that had been built. There were twenty-seven wall locations identified through this spreadsheet. NDOT personnel identified a Lovelock MSE wall as the first MSE wall to be built in Nevada. A review of the contract bidding history was completed back to 1973, which is one year prior to the first Lovelock MSE wall. Seven more MSE walls were identified through this review. At this point thirty-four wall locations had been identified. It was thought that this was a fairly comprehensive list. However, several interviews with NDOT personnel who reviewed the list helped to identify other walls that were not included in the initial list. These interviews along with observations while conducting site visits to wall locations on the list helped increase the number of NDOT wall locations to a total of forty-one.

Once the wall locations were identified and contract numbers found it was possible to start collecting the pertinent information regarding corrosion issues. Through an extensive literature review of the factors that can affect corrosion rates and a review of design methodologies, several key elements were collected for each wall location. These details and the information collected were found by reviewing archived documents in microfilm, searching through boxes of construction reports and design information, and sorting through files of testing data. All of the data collected was found in the Carson City records office, Bridge Division storage shed, and Materials Division storage files. Some of the documentation that was found for some walls could not be found for other walls either because the documents had been destroyed, misfiled, or were just missed by the author during the sifting of thousands of pages of documents. More on this can be found in later sections of this chapter.

5.1.1 Information Collected

The information that has been collected for the wall locations included in this database can be divided into several main groups. These include general NDOT information, physical location, physical characteristics, geotechnical and structural design information, and materials testing data. The fields included in each of these groups of information have been detailed below. Also included in the discussion of the fields are the justifications for assumptions made regarding the infilling of data in instances where proof could not be found in the data collection. If the data in the fields are based on assumptions the data is followed with a (?) to specify that the data was not found but can be assumed with a fair amount of certainty.

- **General NDOT information**
  - **Contract number** – This is a number that is given to each project and a useful identifier when trying to find information from archived records and from Bridge and Materials Divisions' files.
  - **Date of drawing** – Each as-built that was found in the records building had a date on the front page of the design drawing package.
  - **Date of contract** – The date signifies the date the contract was signed. It can be assumed that construction on the project started after this date. This date was not always available and MSE wall construction dates were difficult to ascertain.

- **District** – Nevada is divided into three districts.
    - District 1 contains the southern portion of the state including Las Vegas and Clark County.
    - District 2 includes the cities of Reno and Carson City and Washoe County.
    - District 3 contains the northeastern portion of the state including the cities of Elko and Winnemucca.
- **Demography** – The walls have been characterized as either urban or rural depending on their location

- **<u>Physical location</u>**
    - **County** – This identifies the county where the wall is located.
    - **Intersection** – A majority of the MSE walls found are centered about intersections. Where possible, a descriptor is used to more closely identify the wall location with respect to major intersections.
- **<u>Physical characteristics</u>**
    - **Number of walls** – This is the number of walls constructed at the wall location. Walls have been counted as individual walls as identified by NDOT as-built drawings. In the case of back-to-back walls the number of walls is counted as two walls.

- **Maximum height** – The maximum height of the walls constructed at each location is considered the exposed height of the wall from finished grade at the toe of the wall to the top of the wall where it meets the coping.

- **Facing area** – This measure is one of the methods of payment for MSE wall construction and is well documented in the bidding documents. It is based on the amount of facing panel area that will be constructed, regardless of wall type.

- **Backfill volume** – This measure is one of the methods of payment for MSE wall construction and is well documented in the bidding documents. It is based on the amount of backfill estimated for MSE wall construction.

- **Wall manufacturer** – There are several wall manufacturers who have wall systems used in Nevada. Identifying the wall manufacturer can aide the identification of soil reinforcement used in the MSE walls. There are cases where there was no documentation identifying the manufacturer or other design issues. If a site was visited during the field visit portion of this research and the panel type was a specific manufacturer's product the field was filled with this assumption. An example of this was described earlier where a wall in District 2 was visited and it was obvious that the wall facing was a Reinforced Earth Company shaped facing (Figure 1).

- **Wall reinforcement type** – The type of soil reinforcement used is a key issue with evaluating whether the wall should be evaluated further for potential of high corrosion. There are typically two types including ribbed steel strips and welded wire fabric (WWF) soil reinforcements. Barmat soil reinforcements are a possible third type, but no walls were found to use this reinforcement type. All of these reinforcement types can also have a galvanized coating.

- **<u>Geotechnical and structural design information</u>**
  - **Design life** – The walls found are permanent structures and it is common to have seventy-five to one hundred year design lives. Although some of the older walls in the database do not have defined design lives in the documentation that was found, the 1986 NDOT Silver Book states that the design life should not be less than seventy-five years. Where this Silver Book edition was referred to in wall documents it was assumed that the design life should be at least seventy-five years.
  - **Galvanization life** – In several of the MSE wall location files the design calculations were found. These calculations included calculated loss rates for the galvanized coatings.
  - **AASHTO standards** – These standards help identify the practices used during the wall construction. Assumptions were made based on dates of drawings and information found for other chronological walls when this information was not found.

- **NDOT standards** – These standards help identify the practices used during the wall construction. Assumptions were made based on dates of drawings and information found for other chronological walls when this information was not found.
- **Retained backfill internal friction angle** – This is a design input parameter for wall stability calculations.
- **Retained backfill unit weight** – This is a design input parameter for wall stability calculations.
- **Reinforced backfill internal friction angle** – This is a design input parameter for wall stability calculations.
- **Reinforced backfill unit weight** – This is a design input parameter for wall stability calculations.
- **Design methodology** – There are only a handful of walls that have specified design methodologies. This can be useful for future wall evaluations.
- **Seismic characteristics** – This set of data includes the seismic acceleration (in % g) and the percentage of peak ground acceleration used in design.
- **Sliding coefficient** – This is an input parameter for design.

- **Materials testing data**
  - **Soil resistivity** – These are results of backfill resistivity measurements conducted by the NDOT materials laboratory using the Nevada T235 test method.
  - **Chloride content (Cl)** – This is the quantity of chloride ions in parts per million (ppm) in the approved backfill materials that may have been used in the MSE wall construction. Until 2004, an unnumbered Nevada test method was used. The current practice is to use AASHTO T-291 to measure the chloride content.
  - **Sulfate content (SO_4)** – This is the quantity of sulfate ions in parts per million (ppm) in the approved backfill materials that may have been used in the MSE wall construction. Until 2004, an unnumbered Nevada test method was used. The current practice is to use AASHTO T-290 to measure the sulfate content.
  - **pH** – This is the measured pH level of the backfill soils approved for use in MSE wall backfill typically using the Nevada T238 test method.

5.1.2 MSE Wall Database

The data fields included in the MSE wall database were first included in a spreadsheet format. Although the data is still in that format for NDOT personnel that prefer to use a spreadsheet program, the data has also been populated into a Microsoft

Access© database. This allows for more capabilities that can aide the user in reviewing important information with respect to each wall location. There are several pieces of information that have been included in the database that have not been included in the spreadsheet. Wall photos from site visits have been attached to the database to aid future investigators with wall identification. Some of the more important notes from the field site visit observations have been included as well. On request of the Bridge Division, a scanned-in PDF version of wall locations collected from the as-built drawings have been attached to the database too. An abbreviated version of the database, with some of the more pertinent details, is included in Table 20.

There are some interesting statistics and conclusions that can be drawn from review of the database of collected MSE wall information. The data that is discussed here will only include thirty-nine of the forty-one wall locations because, as detailed previously, one of the wall locations has been decommissioned, and there are two wall contracts (3148 and 3292) which represent the same location where walls have not been completed. The wall construction dates range from 1974 to present. There are a total of 154 walls located at these thirty-nine locations. Of the thirty-nine locations thirty-one have been located in urban locations and eight are in rural areas. Steel strips and welded wire mesh are the two different types of soil reinforcements. All of the walls using steel strips have also been identified or are assumed to have galvanized coatings. These walls, at the fifteen wall locations, are Reinforced Earth Company walls and the galvanization practice has been used historically by the company. The welded wire fabric walls, used at nineteen locations, include seventeen walls that incorporate galvanized coatings and two walls that do not have galvanized coatings.

Some of the more pertinent information regarding the MSE walls can also be evaluated by district. There are three districts in Nevada. In District 1, the southern Nevada district that includes Las Vegas, there are eighteen wall locations with a total of ninety walls at these locations. These wall construction dates range from 1981 to present. All of the wall locations have been classified as urban locations. Of the eighteen wall locations, there are five known wall manufacturer/designer groups. Three of the eighteen wall locations are not readily identified as any of the five wall manufacturer groups because the historical literature did not identify them and the wall locations were not visited by the author. Six of the wall locations known were constructed using Reinforced Earth Company Wall design and materials. SSL Company designed and provided panels for four of the wall locations, while VSL Corporation and Hilfiker Retaining Walls each were used at two wall locations. Foster Geotechnical and Retained Earth design and manufactured panels were used at one wall location. Of the known wall types, six used galvanized steel strips, eight used galvanized welded wire fabric (WWF), and one used welded wire fabric that was not galvanized. The one wall where the WWF was not galvanized was the Flamingo wall, which has been discussed in some detail in Chapter 4. The maximum wall heights range from fifteen feet to more than forty-five feet for the wall locations in District 1.

The second district, District 2, includes Reno, Sparks, and Carson City. The first MSE wall constructed in Nevada at Lovelock is also located in this district. The dates of construction for these walls range from 1974 until present time, with several walls currently under construction. There are nineteen wall locations constructed in District 2 with a total number of walls at all of these locations summing to sixty-one. There is a repeat of walls in Contracts 3148 and 3292, so the more recent contract is included in the count. There are thirteen wall locations that have been classified as urban and six locations that have been classified as rural. Of these nineteen wall locations there are nine locations with Reinforced Earth Company walls, three locations with VSL Corporation walls, three with SSL Corporation walls, and two Hilfiker Retaining Wall Company walls. Two of the wall types are unknown due to lack of data. The soil reinforcement types include nine walls with galvanized steel tie strips, eight with galvanized welded wire fabric, and one wall with welded wire fabric that was not galvanized. One wall location soil reinforcement type was not identified. The maximum wall heights range from eight feet to sixty-four feet for the wall locations in District 2.

District 3, which includes the cities of Winnemucca and Elko, has two wall locations. One, located in Winnemucca, is classified as urban and the other is classified as rural. There are a total of three walls at these two locations. Two of the walls located in Winnemucca are Hilfiker Retaining Wall Company walls that used galvanized welded wire mesh. The other wall location was not identified as any specific company. Discussions with NDOT personnel identified the facing materials as modular or segmental blocks, but it was not clear what type of metallic soil reinforcement was used. The maximum wall heights range from twelve feet to fourteen feet for the wall locations in District 3.

The widespread use of galvanized coatings on the soil reinforcing materials can give the illusion that there should not be many walls that will have experienced corrosion rates similar to those seen in the Flamingo walls. However, as seen in the galvanized Cheyenne wall, the caveat is that the backfill soil needs to be mildly corrosive to non-corrosive in its electrochemical behavior for standard rates of corrosion. On the other hand, if the backfill is corrosive much higher rates of corrosion are expected. With this understanding, a collection and evaluation of data regarding backfill materials and their characteristics was compiled. The materials testing results are not included in the database, due to the difficulty in detailing the large datasets found for each MSE wall location. These have been included in a spreadsheet instead.

5.1.3 Materials Testing Spreadsheets

The Materials Division at NDOT has performed tests on backfill that was submitted for acceptance purposes during the MSE construction process. A majority of the MSE wall locations have test results that have been included in this spreadsheet. There are only a handful of contracts that did not have electrochemical testing data. Out of the thirty-nine distinct and existing MSE wall locations, backfill data was collected for twenty-nine of them. It is interesting to note that electrochemical testing data was not found at all prior to Contract 1918 (1982). While both approved and rejected testing results were collected when found for each contract, only the approved data has been included in this spreadsheet. This is based on the assumption that only approved backfill was used in MSE backfill. There may be an issue with this assumption as discussed in the statistical analysis section of the Flamingo wall backfill in Chapter 4.

The fields of data that were included in this section include soil resistivity, chloride and sulfate contents, and pH level. Soil resistivity test data has been the primary focus of the analysis and in the prioritization process used to select MSE walls that should be further investigated for corrosion problems. There are several reasons that resistivity is focused on so heavily. A correlation between the Nevada and AASHTO test methods has been developed that allows for conversion of historic Nevada test data into AASHTO test data. The correlation shows that the Nevada test method consistently over-predicts the soil resistivity compared to the AASHTO test method, which is not conservative. More on this correlation has been discussed in Chapter 3. Another reason for the focus on resistivity is that it has been shown to be an effective tool to predict the corrosive nature of backfill soils (Elias 2000). The chloride and sulfate contents are useful as well, but the resistivity test is an indirect measure of the overall salt content.

One other useful development from this collection of data is the ability to look at backfill source trends. The data presented in Chapter 6 can also be used to show that there are several pits that consistently supply backfill materials that have passing soil resistivity results with the Nevada test method, but would likely not have passing tests if the AASHTO test method was used. While this is not to say that the source pits cannot provide backfill that is not aggressive it does show that there are trends in areas of source materials that the Materials Division may want to be aware of or keep in mind for future projects.

5.2 Limitations of Data Collection

The wall locations identified in this database may not include all of the wall locations that NDOT currently has in Nevada. Significant effort went into identifying as many walls and as many characteristics of those walls as was reasonably possible. Other walls may be added to this list, based on other observations and future contracts. However, the author believes that a significant majority of the existing wall locations have been identified.

Chapter Six
Prediction of Corrosion Behavior of Other Nevada MSE Walls

One of the main tasks of this research is to identify if there is potential for higher than expected rates of corrosion at other MSE walls in Nevada. As was seen in Chapter 4, there are two walls that have been identified as having detrimental rates of corrosion. These two case studies suggest that a careful review of other walls is needed to ascertain safety of those walls. In both case studies the MSE walls were only identified after accidental discovery of corroded soil reinforcements. This is not the most effective practice in identification of other MSE walls with high rates of corrosion. There are some methods that have been proposed over the years that aide in the evaluation process. However, with thirty-nine wall locations and 154 walls at those locations, a systematic approach is much more useful. The following sections provide some guidance with respect to evaluating existing MSE walls with the data that is readily available. This will be a major first step in selecting candidate sites for future investigations. The following sections make it clear that the two sites that have been investigated recently are not likely to be the only sites that have experienced significant corrosion issues.

6.1 Evaluation of Historical Nevada MSE Backfills

One of the activities that was conducted in this research detailed in Chapter 5 involved collecting background data for the MSE walls owned by NDOT, located across Nevada. In this background research the Materials Division records were reviewed for backfill test data with respect to electrochemical testing. This data is available because during the construction process the contractor constructing the MSE walls submits representative samples of backfill materials for acceptance testing. Approved backfill sources are ones that have met all of the specifications outlined in the NDOT Silver Book. Once the backfill source is approved the material from that source can be used in the construction of the MSE walls. The data from these acceptance tests was found and collected for thirty of the thirty-nine wall locations. The analysis of these results included the evaluation of soil resistivity, chloride and sulfate contents, and pH measurements. However, pH measurements have been addressed in other chapters and will not be revisited.

Though the focus of this evaluation is on the electrochemical test data, attention has been given mainly to soil resistivity measurements. The main reason is that this test method estimates the total salt content and can be used as a good approximation for soil corrosion potential. In all of the contracts reviewed the Nevada T235B soil resistivity test method was used to evaluate the soil resistivity. However, the sacrificial steel that is required for a specified design life is calculated by using the AASHTO designated corrosion loss model. The distribution of soil resistivity measurements using the Nevada test method is presented in Figure 59. As has been discussed in several of the previous chapters, the Nevada test method over predicts the soil resistivity compared to the AASHTO T-288 soil resistivity test method. The significance of this over prediction can be evaluated in Figure 59, where a line indicates AASHTO equivalent 3,000 ohm-cm. There are 44 of the 118 (37%) approved backfill tests that fall below the equivalent AASHTO minimum resistivity. Because of the over predictive nature of the Nevada test method the correlation between these two test methods, discussed in Chapter 3, has been applied to these data sets as well; thus allowing an equivalent AASHTO soil resistivity estimate. It is apparent from this figure that there are a significant number of approved backfills, which are considered corrosive to very corrosive may have been used as the MSE wall backfill in Nevada.

The AASHTO resistivity estimates (Equation 3.1) of the thirty wall locations where test data was found are presented in Figure 60. The maximum, minimum, and average values reported are presented so that the range of values can be seen. The wall construction has also been divided into decades so that one can evaluate which walls are older. While the test data represents the samples that were submitted and approved for use, the data does not necessarily represent the backfill that was used, as seen in the statistical analysis of the Flamingo and Cheyenne backfill data. A contractor could supply several samples for source approval. Many of those sources can be approved, but the contractor may only use a few of them. What is unknown is, once the approval process is completed whether the contractor used the approved soils with higher or lower resistivity values. These results can be used as a starting point for the wall prioritization process. It is also interesting to note that for the Flamingo and Cheyenne data, both approval data from prior to construction and subsequent on-site investigation data have been included as a frame of reference for potential variation in backfill characteristics. These sets of data can be used as anchoring points when evaluating other walls because both of these walls had approval data during construction, while their more recent on-site investigation data falls significantly below the original data.

The contracts have also been divided by respective districts to evaluate any patterns in low resistivity values. There are only two wall locations that have been constructed in District 3, so they are included only in Figure 60. The wall locations in District 1, including seventeen wall locations, are presented in Figure 61. With the exception of two contracts, these wall locations have soil resistivity ranges that are centered about the minimum resistivity specification (88%). There are four wall locations (24%) that do not have a single resistivity measurement above the 3,000 ohm-cm minimum limit. Therefore, from this evaluation, there are a number of walls that may have lower resistivity values than is recommended by NDOT and AASHTO when using the AASHTO T-288 method.

In Figure 62, District 2 walls are presented. There are a total of twelve walls that have available test data. Three of these walls have resistivity values that may be at, or lower than the accepted resistivity requirements (25%). There is only one wall (8%) where all of the test data falls below the minimum requirement of 3,000 ohm-cm. Comparing the two districts, it appears that District 1 has a significantly higher selection of walls that may have lower resistivity values than is desired. A corrosive environment in MSE backfill can be created due to the existence of lower soil resistivity properties. This data evaluation can be used as a starting point when selecting candidate walls for future investigations. There are nine of the thirty walls (excluding the Flamingo and Cheyenne investigated walls) that have average resistivity values less than the 3,000 ohm-cm limit. These walls are included in Table 21. The data included in this table presents the salt content data ranges as well, in order to further fine-tune the selection process.

The test results from the soluble salts, including chlorides and sulfates have been included in Figures 63 and 64, respectively. A vast majority of the test samples fall below the minimum limits recommended relative to soluble salts. Only twenty-five contracts have been included here. Five of the thirty contracts with resistivity data did not have soluble salt data. This is because the resistivity values were greater than 5,000 ohm-cm. The current practice is to waive the soluble salt testing requirements if the resistivity is above this value. Therefore, five contracts do not have this data for any of their approved samples. Only one wall location, not including the Flamingo walls, has chloride values greater than the current maximum 100ppm limit while two wall locations (excluding the Flamingo walls) are above the sulfate maximum limit of 200ppm. These walls also had very low resistivity values, as is seen in Table 21.

From these figures a better prediction can be made regarding which walls should be evaluated in the continuing Phase II study of MSE wall corrosion. Comparing the data of the Flamingo and Cheyenne case histories with the data of other contracts, it is possible to evaluate the likelihood of other walls that may be experiencing higher rates of corrosion. The list of wall locations in Table 21 presents a starting point for this evaluation.

The issue of wall age is important. More concern should be given to older walls that with higher rates of corrosion will have serious limitations on their service lives. On the other hand, newer walls offer the opportunity to be evaluated with less concern for short term failure, where monitoring techniques would provide estimates of service life reduction before issues related to mitigation are required.

As was seen in the Flamingo walls, twenty years can be considered the service life of a wall constructed with aggressive soils. There are two other wall locations in Las Vegas that are older than the Flamingo walls. One of these walls is on the list of potential walls to be investigated because of the potentially aggressive backfill. The other wall was constructed prior to some of the materials testing requirements that were imposed in backfill approval processes. Due to the apparent aggressiveness of soils in Las Vegas, the walls constructed (year 1981) under Contract 1916 should be considered as well. Both of these wall locations are over twenty-five years old and the approval test results for backfill used in Contract 1918 are considered corrosive.

Soil reinforcement type should also be a consideration when prioritizing which MSE wall locations to investigate. The walls from Table 21 have been included in Table 22 which details the soil reinforcement type at each wall location. This table also includes Contract 1916, which was identified above as having a high probability for higher than anticipated rates of corrosion. By chance four wall locations with galvanized tie strip reinforcements and four wall locations with galvanized WWF reinforcements have been identified. There are two wall locations where the soil reinforcement types have not been identified. Eight of the ten wall locations are located in the Las Vegas area. Soil reinforcement type is a factor in corrosion mechanisms since one reinforcement type may perform better than another.

The three MSE wall locations that were constructed prior to any of the Las Vegas walls include two wall locations on State Route 431 on the Mount Rose Highway and the first walls constructed in Nevada in Lovelock. None of these contracts included materials testing data for the backfills used. The two wall locations on State Route 431 were likely constructed using decomposed granite backfill because of material availability in that area. The walls constructed in Lovelock in 1974 were investigated a few years ago. No personnel at NDOT could locate the report or conclusions from the report regarding this investigation. However, their general consensus was that the walls and soil reinforcements were found to be in good condition. From these findings it can be assumed that the walls constructed prior to Contract 1916 should not be the focus for further immediate investigation.

There are two walls that should be included in the list for monitoring efforts. The first is the other Flamingo walls where corrosion issues have already been identified and monitoring stations have been installed. The second wall location should be the remaining Cheyenne walls. The removed wall section had high rates of corrosion. Therefore it is likely that other walls at that location should be evaluated. Table 23 ranks all of the walls that had soil resistivity average values that fell below the minimum soil resistivity requirements when the AASHTO correlation was used. Once the walls have been ranked a work plan can be developed. Suggested practices are detailed below. Recommendations have been included in the final chapter regarding which methods the author suggests for each wall in future investigations.

6.2 Methods for Future Evaluation of Existing MSE Walls

Once the MSE wall locations are selected there are several methods of analysis that can be used. Direct observation of the outside of the wall is not likely to be useful. During the site visits conducted by the author it was observed that even walls such as the Flamingo walls did not show outward signs of distress that could be directly related to corrosion. There were instances where walls were not exactly vertical, but these could be cases created during construction. There are four groups of evaluation methods that will be identified in this research. Each of these methods has its own usefulness, but some will be more costly than others. The four groups of evaluation methods for existing walls include representative backfill soil testing, installation of non-stressed soil reinforcements, nondestructive monitoring methods, and destructive direct observational methods.

6.2.1 Representative Backfill Soil Testing

A review of the electrochemical properties measured at the Flamingo and Cheyenne MSE wall case studies shows that the backfill that was used in MSE wall construction is statistically different from the approved backfill for several of the properties evaluated. When this understanding is extrapolated, there may be other walls that could be in similar conditions. After selecting wall locations that warrant further investigation, the first step in analysis is to identify a more accurate representation of the electrochemical and physical properties of the soils of in-place backfills. While the methods of evaluating the electrochemical properties have been discussed in previous chapters, the methods for obtaining a statistically significant number of samples have not been introduced. With the variation of soil characteristics that is likely to occur in the backfill for these walls, it is important to obtain a representative number of samples that will be needed to address the variability that can be expected.

A power analysis was used to identify the number of samples required to obtain representative sampling (Fernandez 2009). The resistivity results from the Flamingo wall data were used because the original approved backfill dataset was found to be statistically different from the 2005 investigation backfill dataset. A coefficient of variation (COV) of each sample set was calculated. The 1985 dataset has a COV of 33.6% and the 2005 dataset has a COV of 106%. The later value is too large to develop any meaningful sampling statements. Because of the large variations in data and the relatively small data sets it was decided to perform a logarithmic transformation on the data and recalculate the COVs. After a logarithmic transform the resulting COVs are 4.47% and 13.9% for the 1985 and 2005 data, respectively. Using these values in a power analysis can yield a number of samples that should be tested in order to have a sample power of 0.80 or greater. The transformed data suggest that nine samples are needed for a sample power greater than 0.80, while twelve samples are required for a sample power greater than 0.90 (Figure 65).

The samples retrieved from the Flamingo MSE wall backfill were obtained from behind all three walls. There are a total of 29,100 square feet of facing and 16,100 cubic yards of backfill used at the three Flamingo MSE walls. The number of samples obtained from the Flamingo MSE walls can be normalized with respect to either of these values in order to identify the number of samples required for statistically meaningful sampling programs at other walls. If the square footage of facing is used, three samples should be collected from each 10,000 square feet of facing for a sample power of 0.80 and four samples should be collected from each 10,000 square feet of facing for a sample power of 0.90.

A general rule of thumb for sample testing suggests that three samples should be the minimum number collected. As was mentioned on several occasions, there are no precision and bias statements to use in evaluation of test results for the AASHTO resistivity method. Therefore, more samples are required in order to ensure that the variability in the backfill and testing procedures are reduced. A reduction in sensitivity has also been introduced in the use of a logarithmic transformation of the data. With these issues of variability and sensitivity, it is recommended that the results from the power analysis are doubled. This author suggests that six samples are retrieved per 10,000 square feet of wall facing. This will reduce the likelihood of misidentifying backfill characteristics that are different from the originally approved backfill.

The sampling of backfill soils will likely be conducted using test boring equipment. The methods of sampling will be similar to those used in current geotechnical soil exploration practices. It should be kept in mind that the active wedge interface starting at a distance equal one third of the wall height behind the wall face is a critical area regarding corrosion. Sampling should be random in location with emphasis given to this critical area. Consideration should also be given to depth into the backfill and location along the walls. Field observations will need to be made regarding sample similarities to other samples collected and types of materials collected. Future statistical evaluation will be required when several samples appear to be dissimilar in gradation and appearance.

Further modification of this specification can be made as the variation in measurements is reduced. In general, an assumption of homogeneity is made regarding the backfill at each wall location. However, there are many wall locations where several walls have been constructed and large quantities of backfill were required. There is a possibility that some of these wall MSE walls may have been constructed with backfill from several sources, thus creating a need to use a block sampling evaluation.

6.2.2 Installation of Non-Stressed Soil Reinforcements

Since 1979, Caltrans has included inspection rods (or coupons) into their MSE wall backfills (Jackura et al. 1987). It was the practice of systematically evaluating these inspection rods that allowed them to identify the Mariposa wall (Chapter 2) that was experiencing higher than anticipated corrosion rates. During the Flamingo investigation there were "dummy" coupons installed into the backfill so that corrosion evaluations could continue (Fishman 2005). One option that can assist in identifying other NDOT MSE walls that are experiencing higher rates of corrosion is to install both galvanized and bare steel inspection rods or coupons that are not part of the structural components of the MSE walls with the provision that they can be removed at specified time intervals. It is suggested that the Caltrans method, which has proved to be successful, be used for the evaluation of corrosion. It is recommended that NDOT develop a monitoring procedure and evaluation as a part of their highway/bridge maintenance program to evaluate the corrosion status of these coupons on a regular basis.

An important issue is that these inspection rods should be evaluated by direct observation. Both galvanized and bare steel samples should be used so that the rate of corrosion of each material can be evaluated. This will also enable the prediction of corrosion of galvanized steel once the protective coating has been depleted. It is important to keep some of the corrosion behaviors presented in Chapter 3 in mind when evaluating the bare steel samples in the early years because of passivity effects. The next section will introduce the use of nondestructive blind measurements, but those methods produce a uniform corrosion rate estimate. As has been seen in the Flamingo and Cheyenne walls, a uniform measurement is not a conservative estimate of what is actually occurring in the backfill. However, the estimates of uniform corrosion rate can be useful in providing an estimate of the average corrosion and can also bracket that estimate.

6.2.3 Nondestructive Monitoring Methods

There are a large number of publications and articles that have addressed the use of indirect measurements of corrosion in MSE backfill. The current FHWA manual on corrosion and degradation of MSE wall soil reinforcements commits a whole chapter to this issue (Elias 2000). There are also other publications that present methods of installation and observation for polarization resistance (PR) monitoring and half-cell potential measurements (Elias 2000). The existing MSE wall soil reinforcements that are under investigation can be connected to these devices in order to make measurements and evaluations. However, there are numerous publications that specify that these measurements need to be carefully analyzed so that interpretations are more accurate. The issue is that there is no baseline reading that was performed at the initial installation of the soil reinforcement (NCHRP 24-28 2007). Without this baseline measurement the person evaluating the measurements has to make an assumption about the original state of the steel reinforcement.

One method of using the indirect measurement techniques is to couple it with the previous method of observation rods and "dummy" coupons. Using direct observations with PR monitoring can be a very useful method of evaluation. The observation rods can be extracted at intervals of several years while the PR monitoring measurements are taken at seasonal intervals. The combination of these two methods can be a powerful tool in corrosion rate estimation.

6.2.4 Destructive Direct Observational Methods

The final of the four general methods for estimating corrosion involves destructive testing. Both the Flamingo and Cheyenne walls were investigated using destructive methods. Advancing test pits into the MSE wall backfill, either at the top or into the facing, is required to make direct observations of the soil reinforcements. This is a method that should be considered as a last resort. After using the previous three methods in concert to identify walls that have experienced higher rates of corrosion, this destructive method can be used to identify the severity of the corrosion. Once the severity is evaluated, mitigation efforts can be undertaken. Proper implementation of the first three methods can reduce the potential possibility that a wall needs to be investigated in this destructive manner.

6.3 Future MSE Wall Investigation Recommendations

There are a number of walls that have presented themselves as potential candidates for future evaluation. Methods for these future evaluations are also presented in this chapter. The evaluation process can be performed in a series of steps, starting with the lowest expense and increasing as the evaluation continues. The first step will be to identify the corrosive nature of the backfill that has been used in the construction of these walls. The method for analysis has been referred to in Section 6.2.1 above. One clear finding from the Flamingo and Cheyenne backfill analyses is that the corrosive nature of the soils used as backfill were not clearly identified during the construction process. The existing backfills need to be evaluated prior to any further wall evaluation.

Once the backfill at a specific wall location is truly identified as corrosive the evaluation should progress to step two, identified in Section 6.2.2, which involves the installation of monitoring devices. This is especially useful for the more recently constructed MSE walls as corrosion assessment of coupons and PR measurements can be monitored over a longer period of time. However, the older walls, constructed in the 1980s should be considered higher risk walls, and should be treated accordingly. It is clear that the existing two Flamingo walls will not have service lives of seventy-five years, even if less conservative estimates of loss are used. Even though the two wall locations in Las Vegas that are older than the Flamingo walls and have been constructed using galvanized soil reinforcements, they are suspect if their backfill soils are found to be corrosive in step one of the investigation.

The more recent walls offer a great opportunity for proactive predictions of corrosion rates and should be treated as such. They should be monitored regularly once their in-place backfills have been evaluated and found to be corrosive. The corrosive nature of Nevada's soils has been clearly documented and the MSE walls that have been constructed should be evaluated further so that potentially catastrophic failures can be avoided in the future.

Chapter Seven
Conclusion and Recommendations

Mechanically stabilized earth walls are a practical solution as retaining structure, and have been incorporated in a large number of NDOT projects resulting in over 150 walls in Nevada. However, as is commonly practiced with other structures, these retaining walls require periodic monitoring and performance evaluations. It appears that corrosion monitoring is an important component in the successful performance of MSE walls. As has been noted several times in this report, exterior evaluations or observations of wall facing during site visits are not a sufficient method for corrosion monitoring. This is especially true because no baseline comparison of deformation measurements exists. Corrosion monitoring can only be conducted by evaluation of the soil and reinforcement conditions behind the wall facing. This is evidenced by the fact that two MSE wall locations (I-515/Flamingo and I-15/Cheyenne intersections) have been found to have high rates of corrosion that directly affect their abilities to perform adequately over their designated service lives. One of the three MSE walls at the Flamingo intersection has been retrofitted with a cast-in-place tie-back wall, at a great expense due to the effects of high levels of corrosion on the soil reinforcements. Only accidental discovery of corroded reinforcements led to these discoveries. Outward observations of these walls showed no signs of distress that would lead to the conclusion that the soil reinforcements were experiencing detrimental loss in cross sectional area due to corrosion. These higher rates of corrosion were produced for both galvanized and bare steel reinforcements demonstrating that galvanized MSE wall reinforcements are also subject to advanced corrosion in Nevada.

The Flamingo and Cheyenne wall corrosion issues are well documented and identified. However, the question now is, how many other walls in Nevada are experiencing higher than anticipated rates of corrosion. This is not a simple question to answer and the investigation attempts to answer this important question. There is significant potential for other walls to have high rates of corrosion because of the unintentional use of aggressive MSE backfill in Nevada. The use of the Nevada T235B test method, which over predicts the soil resistivity, has allowed the use of more corrosive soils in Nevada MSE walls. The Nevada T235B test method measures the conductivity of water from a saturated backfill soil solution. This method of resistivity measurement is significantly different from the AASHTO T-288 test method which uses a soil box to measure backfill resistivity directly. A correlation between the Nevada T235B and AASHTO T-288 resistivity test methods shows that the Nevada test method is not conservative with respect to identifying aggressive soils. Because of this, the corrosive nature of the backfill used in other Nevada MSE walls has been reevaluated. The results in Chapter 6 show that there are at least nine more MSE wall locations that may have been constructed with aggressive soils. The use of these aggressive soils directly affects the internal stability of these walls and these walls need to be investigated further.

The originally approved backfill test data for the Flamingo MSE walls is misleading in its characterization of the aggressiveness of the backfill, which was subsequently demonstrated by MMCE. This provides the need for the immediate evaluation of other Nevada walls. This is substantiated by the statistical analysis of backfill properties in this report. The statistical analysis focused on the electrochemical properties of the initially approved backfill for the Flamingo and Cheyenne case studies and compared that to the properties of the backfill that was actually used in the construction of the walls. The results from this analysis show that the characterization of backfill approved during construction did not effectively predict the corrosive nature of the backfill that was used in MSE wall construction. Because this method of backfill approval has been widely used in Nevada, this practice needs reevaluation as this may have occurred at other Nevada wall locations. Statistical methods for obtaining representative samples are presented so that effective characterization that includes the variability in corrosive material properties can be accounted for.

Recommendations

The recommendations for the future can be divided into two main categories. It is recommended that existing MSE walls in Nevada be evaluated for the potentially detrimental effects of corrosive backfill. It is also recommended the newly adopted practices by NDOT regarding the approval of MSE backfill (production testing) be included as a requirement in future MSE wall construction. The support for these recommendations has been developed within this report and will be summarized below.

Two MSE wall locations with high rates of corrosion were identified completely by accident. It is now clear that there are other walls in the NDOT inventory that are likely to be in the same condition. Twelve of the best candidate wall locations have been identified in Chapter 6 (Table 23). An investigation of these MSE walls should include the following four recommendations:

1. Representative backfill soil testing – all walls should be evaluated to ensure proper characterization of the backfill (i.e. project testing) that was used during construction (Section 6.2.1);

2. Installation of non-stressed (dummy) soil reinforcements in newer walls and as needed in most critical existing walls (as identified in this report – see Table 23) – reinforcement coupons should be installed so that baseline loss measurements can be estimated (Section 6.2.2);

3. Nondestructive monitoring methods – monitoring of corrosion loss, especially including the Flamingo walls, which have monitoring stations already, should be conducted (Section 6.2.3). It is recommended that NDOT develop a monitoring procedure and evaluation as a part of their highway/bridge maintenance program to evaluate the corrosion status of these coupons on a regular basis (e.g. Caltrans procedure).

4. Destructive direct observational methods – walls that are found to have aggressive backfills (based on post-construction electrochemical testing of backfill soils) should be investigated further for direct observation of soil reinforcements (Section 6.2.4).

A combination of these four approaches would have a significant impact on the safety of Nevada walls. The ability to monitor corrosion rates throughout the design life of an MSE wall is strongly recommended given the history and findings of MSE wall corrosion studies and the aggressive nature of the soils in Nevada. This proactive approach will give NDOT the ability to prevent failures in its MSE walls caused by internal stability due to high rates of corrosion.

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Table 1. List of MSE Sites with Detailed Data (NCHRP 24-28 2007)

State	# of Sites	Nondestructive Testing	Direct Physical Measurements	Range of Dates of Construction	Range of Backfill Conditions
California	29	No	Yes	1972-1992	Poor to Good
Florida	8	Yes	No	1979-1996	Good
Georgia	11	Yes	Yes	1974-1990	Poor to Good
Kentucky	4	Yes	No	1979-1993	Good
North Carolina	24	Yes	No	1990-Present	Good
Nevada	1	Yes	Yes	1985	Good
New York	5	Yes	No	1980-2005	Poor to Good

Table 2. AASHTO and NDOT Historical Electrochemical Specifications*

Specification	Pre-1986[a]	1986	1992	1996 to Present	
	NDOT	NDOT 1986 Edition	AASHTO 15th Edition[b]	NDOT 1996 & 2001 Editions	AASHTO 16th, 17th, & 2007 LRFD Editions
pH	6.4 to 9.5	5 to 10	5 to 10	5 to 10	5 to 10
Resistivity (ohm-cm)	1,000 min.	3,000 min.	3,000 min.	3,000 min.	3,000 min.
Chlorides (ppm)	500 max.	200 max.	50 max.	100 max.	100 max.
Sulfates (ppm)	2,000 max.	1,000 max.	500 max.	200 max.	200 max.

*Respective Standard Specifications for Highway Bridges (AASHTO) and Standard Specifications for Road and Bridge Construction (NDOT Silver Book)

[a] There are no references to retaining walls in NDOT editions before 1986, these requirements were found in material test data (Contract 1918, July 1982)

[b] There are no references to MSE walls prior to this AASHTO edition

Table 3. Electrochemical Specifications by State*

State/Agency	pH	Resistivity, R (ohm-cm)	Chlorides (ppm)	Sulfates (ppm)
Nevada (NDOT 2001)	5 to 10 (Nevada T238)	R≥5,000 3,000≤R<5,000 (Nevada T235)	Waived <100 (AASHTO T 291)	Waived <200 (AASHTO T 290)
California (Caltrans 2006)	5.5 to 10 (CA 643)	R≥2,000 (CA 643)	<250 (CA422)	<500 (CA 417)
Oregon (Oregon 2008)	5 to 10 (AASHTO T 289)	R≥5,000 3,000≤R<5,000 (AASHTO T 288)	Waived <100 (AASHTO T 291)	Waived <200 (AASHTO T 290)
Utah (Utah 2008)	6 to 9 (AASHTO T 289)	R≥5,000 3,000≤R<5,000 (AASHTO T 288)	Waived <100 (AASHTO T 291)	Waived <200 (AASHTO T 290)
Arizona (Arizona 2008)	5 to 10 (Arizona 236B)	R≥2,500 (Arizona 236B)	<100 (Arizona 733A)	<200 (Arizona 736A)
AASHTO (2007)	5 to 10 (AASHTO T 289)	R≥5,000 3,000≤R<5,000 (AASHTO T 288)	Waived <100 (AASHTO T 291)	Waived <200 (AASHTO T 290)

*Approved test methods are in parentheses below each specification.

Table 4. Effect of Resistivity on Corrosion (Elias 2000)

Aggressiveness	Resistivity (ohm-cm)
Very Corrosive	<700
Corrosive	700 – 2,000
Moderately Corrosive	2,000 – 5,000
Mildly Corrosive	5,000 – 10,000
Non-corrosive	>10,000

Table 5. Electrochemical Test Methods (Fishman 2005)

Test Method	Laboratory			
	NDOT 2005	Terracon Las Vegas, NV	Terracon Sparks, NV	Geotechnics, Inc
pH	NDOT T238A or AASHTO T-289	AWWA 4500H	AASHTO T-289	AASHTO T-289
Resistivity	NDOT T235B or AASHTO T-288	ASTM G57	AASHTO T-288	AASHTO T-288
Chloride Content	NDOT Method	AWWA 4500 Cl B	EPA 300	CAL 422
Sulfate Content	NDOT Method	AWWA 4500 SO_4 E	EPA 300	CAL 417

Table 6. Flamingo Summary Statistics from Diameter Loss Calculations (Based on 20-yr Life)

Flamingo Summary Statistics of Diameter Loss Measurements						
Descriptive Statistic		**Diameter (in.)**	**Area Loss* (in^2)**	**Estimated Corrosion (yrs)**	**Corrosion Severity Ratio (306µm/side Expected)**	**Estimated Radial Corrosion Rate (µm/yr)**
	Mean	0.209	0.0319	95.4	3.69	56.5
	Median	0.222	0.0310	80.2	3.15	48.3
	Standard Deviation	0.067	0.0197	68.6	2.79	42.7
	Sample Variance	0.005	0.0004	4705	7.78	1821
	Standard Error	0.004	0.0012	4.28	0.168	2.57
	Count	275	275	257	275	275
	Range	0.278	0.0721	282	11.5	177
	Minimum	0.026	-0.0028	0.847	-0.249	-3.81
	Maximum	0.304	0.0692	282	11.3	173
95% Confidence Interval	**Lower Bound**	0.201	0.0295	87.0	3.36	51.4
	Upper Bound	0.217	0.0342	104	4.02	61.5
	84th Percentile	0.276	0.0516	164	6.48	99.1

* Area loss calculations are based on a nominal original cross sectional diameter of 0.298 inches, as specified by Hilfiker Retaining Walls

Table 7. Flamingo Power Loss Equation ($P=kt^n$) Values for Constants (Assuming n=0.80)

Statistic	**Loss Measurement "P" (μm)**	**Parameter "k"**
Mean	1129	103
Upper 95% Confidence Interval	1230	112
Lower 95% Confidence Interval	1028	94
Median	965	88
84^{th} Percentile	1983	180

Table 8. Caltrans 1984 Design Criteria Specifications for MSE Backfill (Jackura et al. 1987)

Backfill Classification	**Resistivity (ohm-cm)**	**Other**
Neutral and Alkaline	> 1,000	pH > 7
Acidic	> 1,000	pH < 7
Corrosive	< 1,000	-
Very Corrosive (Not Included in Evaluation)	< 1,000	Cl > 500 ppm, SO_4 > 2,000 ppm

Table 9. 1985 Approved Backfill with Specification Comparisons

Backfill Source	1986 Specification Comparison				2007 Specification Comparison			
	Resistivity (ohm-cm)		Chlorides (ppm)	Sulfates (ppm)	Resistivity (ohm-cm)		Chlorides (ppm)	Sulfates (ppm)
	Nevada Method	AASHTO Converted*			Nevada Method	AASHTO Converted*		
Wells Cargo	7194	4449	50	0	7194	4449	50	0
NV Rock & Sand	3800	2406	199	123	3800	2406	199	123
NV Rock & Sand	3509	2228			3509	2228		
Apex Plant	6289	3909			6289	3909		
Apex Plant	7042	4358			7042	4358		

Shaded cells did not pass.

Table 10. 2005 Backfill Test Results with Resistivity Test Methods (Based on Fishman 2005)

Backfill Sample Location (Station)	Resistivity (ohm-cm)		Chlorides (ppm)	Sulfates (ppm)
	Measured	Test Method		
Reported by NDOT (2005)				
MSE Fill (152+10)	4388	Converted*	30	0
MSE Fill (155+25)	677	Converted*	30	600
MSE Fill (152+87)	3506	Converted*	20	542
TP-4B	1247	AASHTO		
TP-6B	1307	AASHTO		
TP-7B	1134	AASHTO		
S-6	1234	AASHTO		
Reported by Terracon – Sparks (2005)				
TP-2B	5200	AASHTO	15	15
TP-3B	420	AASHTO	15	380
TP4-B			15	1100
TP-5B			78	4600
TB-5B			83	140
TP-6B			15	160
TP-7B			15	340
S1	450	AASHTO	15	1400
S2			15	430
S3	410	AASHTO	20	300
S4	420	AASHTO	15	390
S5	420	AASHTO	15	470
S11			19	3700
S12			15	910
S13			18	7500
S14			25	2900
S15			15	240
S16			15	3000
S17			230	6900
Reported by Terracon - Las Vegas (2005)				
B-1 D=10'	1950	ASTM	50	3740
B-1 D=20'	5200	ASTM	75	1238
B-2 D=15'			75	660
B-2 D=25'			100	1513
B-3 D=5'			100	8773
B-3 D=15'	3000	ASTM		
B-3 D=20'			225	9075
B-5 D=5'	1300	ASTM		
B-5 D=30'	585	ASTM	500	9625
Reported by Geotechnics (2005)				
S9	7800	AASHTO	70	93

*Calculated from correlation using Eq. 3.1.

Table 11. 2005 Backfill Test Results with Specification Comparisons

Backfill Sample Location (Station)	1986* Specification Comparison			2007 Specification Comparison		
	Resistivity (ohm-cm)	Chlorides (ppm)	Sulfates (ppm)	Resistivity (ohm-cm)	Chlorides (ppm)	Sulfates (ppm)
Reported by NDOT (2005)						
MSE Fill (152+10)	4388	30	0	4388	30	0
MSE Fill (155+25)	677	30	600	677	30	600
MSE Fill (152+87)	3506	20	542	3506	20	542
TP-4B	1247			1247		
TP-6B	1307			1307		
TP-7B	1134			1134		
S-6	1234			1234		
Reported by Terracon – Sparks (2005)						
TP-2B	5200	15	15	5200	15	15
TP-3B	420	15	380	420	15	380
TP4-B		15	1100		15	1100
TP-5B		78	4600		78	4600
TB-5B		83	140		83	140
TP-6B		15	160		15	160
TP-7B		15	340		15	340
S1	450	15	1400	450	15	1400
S2		15	430		15	430
S3	410	20	300	410	20	300
S4	420	15	390	420	15	390
S5	420	15	470	420	15	470
S11		19	3700		19	3700
S12		15	910		15	910
S13		18	7500		18	7500
S14		25	2900		25	2900
S15		15	240		15	240
S16		15	3000		15	3000
S17		230	6900		230	6900
Reported by Terracon - Las Vegas (2005)						
B-1 D=10'	1950	50	3740	1950	50	3740
B-1 D=20'	5200	75	1238	5200	75	1238
B-2 D=15'		75	660		75	660
B-2 D=25'		100	1513		100	1513
B-3 D=5'		100	8773		100	8773
B-3 D=15'	3000			3000		
B-3 D=20'		225	9075		225	9075
B-5 D=5'	1300			1300		
B-5 D=30'	585	500	9625	585	500	9625
Reported by Geotechnics (2005)						
S9	7800	70	93	7800	70	93

Did not pass.

Table 12. (Pr> |t|) of LSMEANS using LSD for Flamingo Original Resistivity Data

	NDOT 1985	NDOT 2005	Terracon – Las Vegas 2005	Terracon – Sparks 2005
NDOT 1985		0.0292	0.0281	0.0029
NDOT 2005	0.0292		0.8366	0.2317
Terracon – Las Vegas 2005	0.0281	0.8366		0.3626
Terracon – Sparks 2005	0.0029	0.2317	0.3626	

Table 13. Wall #2 and #3 Stability Analysis Characteristics

Assumed Details Common to Walls #2 & #3	Values Used in Analysis
Retained Soil Unit Weight (γ)	120 pcf
Cohesion	0
Internal Friction Angle	34°
Facing Height	2 feet
Longitudinal Bar Spacing	1.6 bars per foot
Yield Stress of Steel Reinforcements (F_y)	70 ksi
Permanent Load Factor (γ_p)	1.35
Seismic Load Factor (γ_{EQ})	1.00

Table 14. Expected Failure Lifetimes for Wall#2 and #3 at Flamingo (C/D ratio < 1)

Load Case	**Wall #2**		**Wall #3**	
	Average Power Loss Model (yrs)	84^{th} Percentile Power Loss Model (yrs)	Average Power Loss Model (yrs)	84^{th} Percentile Power Loss Model (yrs)
Static	42	35	39	27
a_{max} = 0.15g	39	32	35	24
a_{max} = 0.21g	38	31	33	23

Table 15. Cheyenne Steel Thickness Loss at Connections for Panel Labeled TR-15 (Top panel with one row of connections)*

	Left Edge	Middle	Right Edge
Near Face	0.126 (65.6%)	0.190 (99.0%)	0.148 (77.1%)
Middle	0.168 (87.5%)	0.144 (75.0%)	0.132 (68.8%)
Away from Face	0.193 (100%)	0.190 (99.0%)	0.193 (100%)

Table 16. Cheyenne Steel Thickness Loss at Connections for Panel Labeled TR-13 (Top panel with one row of connections)*

	Left Edge	Middle	Right Edge
Near Face	0.206 (107%)	0.068 (35.4%)	0.128 (66.7%)
Middle	0.137 (71.4%)	0.130 (67.7%)	0.169 (88.0%)
Away from Face	0.112 (58.3%)	0.129 (67.2%)	0.189 (98.4%)

* Strip measurements are oriented by looking at the front of the facing. The middle edge is measured from left to right.

Table 17. Cheyenne Summary Statistics from Thickness Loss Calculations (Based on 9-yr Life)

Cheyenne Summary Statistics of Cross Sectional Loss Measurements							
Descriptive Statistic		**Area (in^2)**	**Area Loss* (in^2)**	**Estimated Corrosion (yrs)**	**Corrosion Severity Ratio (54µm/side Expected)**	**Estimated Overall Corrosion Rate/Side (µm/yr)**
	Mean	0.295	0.0573	39.4	6.83	41.0
	Median	0.305	0.0475	34.9	5.66	34.0
	Standard Deviation	0.0396	0.0396	21.9	4.73	28.4
	Sample Variance	0.00157	0.00157	480	22.3	804
	Standard Error	0.00398	0.00398	2.21	0.475	2.85
	Count	99	99	98	99	99
	Range	0.170	0.170	99.7	20.2	121
	Minimum	0.183	-3.99 E-05	0.107	-0.00733	-0.0440
	Maximum	0.352	0.170	99.8	20.2	121
95% Confidence Interval	**Lower Bound**	0.287	0.0494	35.0	5.90	35.4
	Upper Bound	0.303	0.0651	43.7	7.76	46.5
	84th Percentile	0.256	0.0969	61.3	11.6	69.3

* Area loss calculations are based on an average original cross sectional area of 0.3524 inches^2, as specified by Reinforced Earth Company design calculations

Table 18. Cheyenne Power Loss Equation ($P=kt^n$) Values for Constants (Assuming n=0.65)

Statistic	Loss Measurement "P" (µm)	Parameter "k"
Mean	369	88
Upper 95% Confidence Interval	419	100
Lower 95% Confidence Interval	318	76
Median	306	73
84th Percentile	624	150

Table 19. Cheyenne Electrochemical Properties for MSE Backfill

Sample		Resistivity (ohm-cm) Nevada T235B	Resistivity (ohm-cm) AASHTO T-288	Chloride Content (ppm)	Sulfate Content (ppm)	pH
1998	Frehner Sloan Pit	9009	5525*	70	0	8.3
	Frehner Sloan Pit	9709	5938*	70	0	8.2
	Chem Star at Apex	3472	2206*	90	0	8.5
	Chem Star at Apex	6173	3839*	90	0	8.5
2008	12' From Face	3470	1477	90	70	8.1
	Near Top Face	6789	3319	30	48	8.2
	Near Rusty Strip	1754	604	210	126	8.0
	Stockpile Sample	4461	1805	70	81	8.2

*Calculated from correlation using Eq. 3.1.

Did not pass.

Table 20. NDOT MSE Wall Database

Contract Number	Date of Drawing	Date of Contract	District	County	Intersection	No. of Walls	Maximum Height (ft.)	Wall Manufacturer	Wall Reinforcement Type
3324	1-Aug-2006	16-Aug-2007	1	Clark	SR 160 and Jones Blvd	6	45.1	Reco	Tie Strips - galv.
3320	13-Jul-2006	21-Nov-2006	2	Storey	USA Parkway Truckee Bridge (Storey County Side)	3	20	SSL	WWF - galv.
3313	20-Jun-2008		1	Clark	I-15 and Cheyenne Ave	1	15	Reco	Tie Strips - galv.
3296	29-Nov-2005	3-Apr-2006	2	Carson	US 50 to Spooner Summit (Underpass)	6	34	Reco	Tie Strips - galv.
3292	28-Jan-2005	6-Nov-2006	2	Washoe	I-580 between Bower's and Mt Rose	8	64.0		WWF - galv.
3290	1-Sep-2005	26-Jan-2006	1	Clark	I-15 and SR 146 Near Southern Highlands Pkwy	6	45.6	SSL	WWF - galv.
3260	8-Jun-2005	28-Sep-2005	1	Clark	Summerlin Pkwy and US 95	3		SSL	WWF - galv.
3254	8-Nov-2004	1-Mar-2005	2	Storey	V&T RR	1 or 2	20+	Hilfiker Retaining Walls	WWF - ungalv.
3237	15-Jul-2004	17-Nov-2004	2	Lyon	Fernley Alt 95 to Alt 50	6	31.25		
3215	20-Apr-2004	12-Apr-2005	1	Clark	US 95 Between Ranch and M.L. King	10	21.3	Reco	Tie Strips - galv.
3189	14-Aug-2003	8-Dec-2003	1	Clark	I-15 and Lamb Blvd.	4	32	SSL	WWF - galv.
3161	24-Mar-2003	3-Sep-2003	1	Clark	US 95 & Rainbow and Summerlin	2	40.3	SSL	WWF - galv.
3154	7-Oct-1999	15-Aug-2003	2	Carson	US 395 and Jumbo Ct	1	9.8	SSL	WWF - galv.
3150	26-Mar-2003	21-Jul-2003	1	Clark	US 95 and Lake Mead Dr.	12	33.6	Foster Geotech - Retained Earth	WWF - galv.
3148	16-Jan-2003	15-Oct-2003	2	Washoe	I-580 between Bower's and Mt Rose	2	64.0	SSL	WWF - galv.
3090	26-Sep-2001	1-Jul-2002	2	Washoe	Spaghetti Bowl (US 395 & I-80)	7	25.3		
3003	23-Nov-1999	21-Mar-2000	1	Clark	I-15 and Sahara to Charleston	3			
2995	3-Nov-1999	24-Feb-2000	2	Carson	US 395 and College Parkway	2	16	Reco	Tie Strips - galv.
2957	25-Feb-1999	2-Jul-1999	2	Washoe	S. McCarran and I-80	2	21.6	Reco (?)	Tie Strips - galv. (?)
2927	4-Nov-1998	2-Feb-1999	2	Washoe	Clear Acre and US 395 **(Does not exist)**	1	11.7	Reco	Tie Strips - galv.

*Reco = Reinforced Earth Company

Table 20. NDOT MSE Wall Database – Continued (*Reco = Reinforced Earth Company)

Contract Number	Date of Drawing	Date of Contract	District	County	Intersection	No. of Walls	Maximum Height (ft.)	Wall Manufacturer	Wall Reinforcement Type
2881	9-Mar-1998	15-Jun-1998	3	Humbolt	US 95 in Winnemucca	2	14.4	Hilfiker Retaining Walls	WWF - galv.
2853	25-Aug-1997		1	Clark	I-15 and Cheyenne Ave	7	37.3	Reco	Tie Strips - galv.
2830	22-Jul-1997		1	Clark	I-15 and US 95	6	21.6		
2795	31-Dec-1996		2	Washoe	I-80 and Pyramid Way	7	28	Reco	Tie Strips - galv.
2779	19-Aug-1996		1	Clark	I-15 and Spring Mountain Rd	15	31		
2776	28-Jun-1996	1-Aug-1996	3	Elko	SR 225 North of Wild Horse Reservoir	1	12		
2674	19-Nov-1995	1-Jan-1996	2	Washoe	US 395 and S. Virginia & Kietzke	2	12	VSL Corp - Retained Earth	WWF - galv.
2593	19-Aug-1993		1	Clark	I-15 and Desert Inn Rd	4	41	VSL Corp - Retained Earth	WWF - galv.
2571	18-May-1993	1-Oct-1993	1	Clark	I-15 and Sahara	1?	28.3	VSL Corp - Retained Earth	WWF - galv.
2567	28-Apr-1993	4-Oct-1993	2	Washoe	US 395 and S. Virginia (btwn Zolezzi & Mt Rose)	2-3	22.5	Reco	Tie Strips - galv.
2260	3-Feb-1988	1-Apr-1988	2	Washoe	I-80 and Sparks Blvd	4	35.8	VSL Corp.	WWF - galv.
2203	2-Apr-1987		2	Washoe	US 395 and Huffaker and Del Monte	4	17.2	Hilfiker Retaining Walls	WWF - galv.
2202	23-Feb-1987	1-Jun-1987	1	Clark	US 95 and Union Pacific RR	2	30	Hilfiker Retaining Walls	WWF - galv.
2121	12-Dec-1985	1-Mar-1986	2	Washoe	US 395 and Plumb Lane (Airport)	3	25	VSL Corp.	WWF - galv.??
2066	5-Nov-1984		1	Clark	US 95 and Flamingo	3	32	Hilfiker Retaining Walls	WWF - ungalv.
1919	7-Jan-1982		2	Pershing	I-80 and Lovelock Main Street	2	13	Reco	Tie Strips - galv.
1918	7-Jan-1982		1	Clark	I-515 and Charleston	4	17.5	Reco	Tie Strips - galv.
1916	23-Jun-1981		1	Clark	I-515 and L.V. Blvd	1	19	Reco	Tie Strips - galv.
1800	21-Mar-1979		2	Washoe	SR 27 and Incline	2	19	Reco	Tie Strips - galv.
1578	27-Apr-1976		2	Washoe	SR 27 and Tahoe Meadows	1	8	Reco	Tie Strips - galv.
1483	14-Nov-1973		2	Pershing	I-80 and Big meadow Ranch Rd	2	19	Reco	Tie Strips - galv.

Table 21. MSE Wall Locations with High Potential to have Significant Corrosion (Excluding Flamingo and Cheyenne)

Contract	Year of Con-tract	District	Resistivity (ohm-cm)			Chlorides (ppm)			Sulfates (ppm)		
			Max.	Min.	Avg.	Max.	Min.	Avg.	Max.	Min.	Avg.
1918	1982	1	2150	1247	1777	190	100	133	600	500	550
2202	1987	1	2872	2471	2671	80	40	60	350	200	275
2203	1987	2	2884	2884	2884	30	30	30	40	40	40
3003	2000	1	3104	2156	2630	80	70	75	0	0	0
3189	2003	1	2348	2348	2348	50	50	50	0	0	0
3215	2005	1	4108	1854	2869	70	30	44	200	114	154
3237	2004	2	3604	1918	2418	70	70	70	0	0	0
3290	2006	1	4300	2191	2901	60	50	55	65	48	57
3324	2007	1	2236	1964	2100	90	65	78	126	85	104

Table 22. MSE Wall Locations with High Potential to have Significant Corrosion Including Reinforcement Type (Excluding Flamingo and Cheyenne Walls)

Contract	Year of Contract	District	Soil Reinforcement Type	Avg. Resistivity (ohm-cm)	Avg. Chlorides (ppm)	Avg. Sulfates (ppm)
1916	1981(?)	1	T.S. galv.	No Data	No Data	No Data
1918	1982	1	T.S. galv.	1777	133	550
2202	1987	1	WWF galv.	2671	60	275
2203	1987	2	WWF galv.	2884	30	40
3003	2000	1	Unknown	2630	75	0
3189	2003	1	WWF galv.	2348	50	0
3215	2005	1	T.S. galv.	2869	44	154
3237	2004	2	Unknown	2418	70	0
3290	2006	1	WWF galv.	2901	55	57
3324	2007	1	T.S. galv.	2100	78	104

T.S. galv.: Galvanized Tie Strips; WWF galv.: Galvanized Welded Wire Fabric

Table 23. Ranking of Nevada MSE Wall Candidate Sites for Future Investigation

Ranking	Contract	Year of Contract	District	Intersection	Soil Reinforcement Type	Avg. Resistivity (ohm-cm)	Avg. Chlorides (ppm)	Avg. Sulfates (ppm)
1	2066	1985	1	US 95 and Flamingo	WWF	1301	48	2435
2	2853	1998	1	I-15 and Cheyenne	T.S. galv.	1801	63	81
3	1918	1982	1	I-515 and Charleston	T.S. galv.	1777	133	550
4	1916	1981(?)	1	I-515 and Las Vegas Blvd.	T.S. galv.	No Data	No Data	No Data
5	2202	1987	1	US 95 and Union Pacific RR	WWF galv.	2671	60	275
6	2203	1987	2	US 395 and Huffaker	WWF galv.	2884	30	40
7	3324	2007	1	SR 160 and Jones	T.S. galv.	2100	78	104
8	3189	2003	1	I-15 and Lamb	WWF galv.	2348	50	0
9	3237	2004	2	Fernley Alt US 95 and Alt US 50	Unknown	2418	70	0
10	3003	2000	1	I-15 and Sahara to Charleston	Unknown	2630	75	0
11	3215	2005	1	US 95 between Ranch and M.L. King	T.S. galv.	2869	44	154
12	3290	2006	1	I-15 and SR 146	WWF galv.	2901	55	57

Shaded Cells do not pass.

Figure 1. MSE Wall Located at South McCarran and I-80 (Contract #2957)

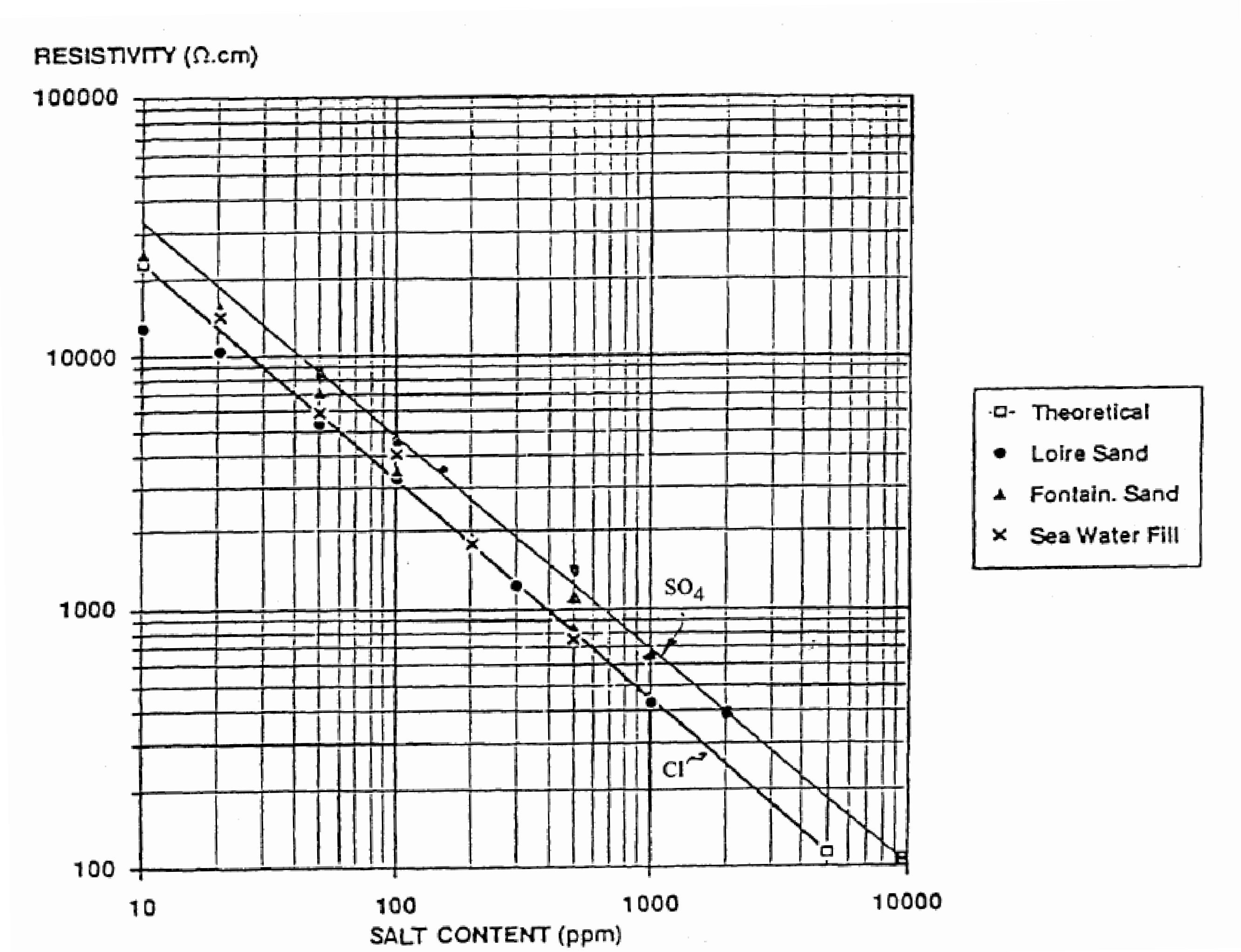

Figure 2. Equivalence in Resistivity of Soluble Salts (Elias 2000)

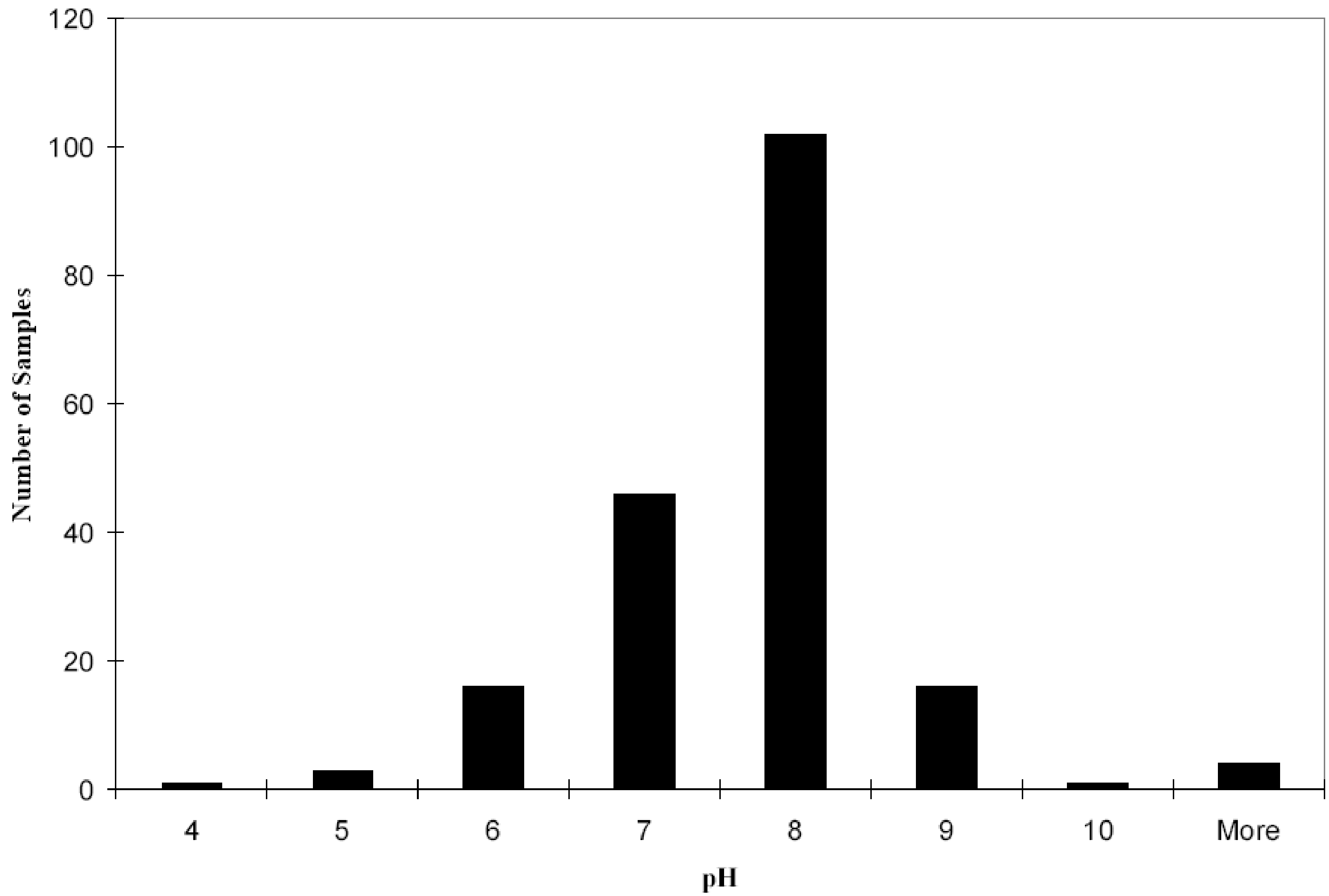

Figure 3. Distribution of pH Measurements for MSE Backfills in the AMSE Survey (AMSE 2006)

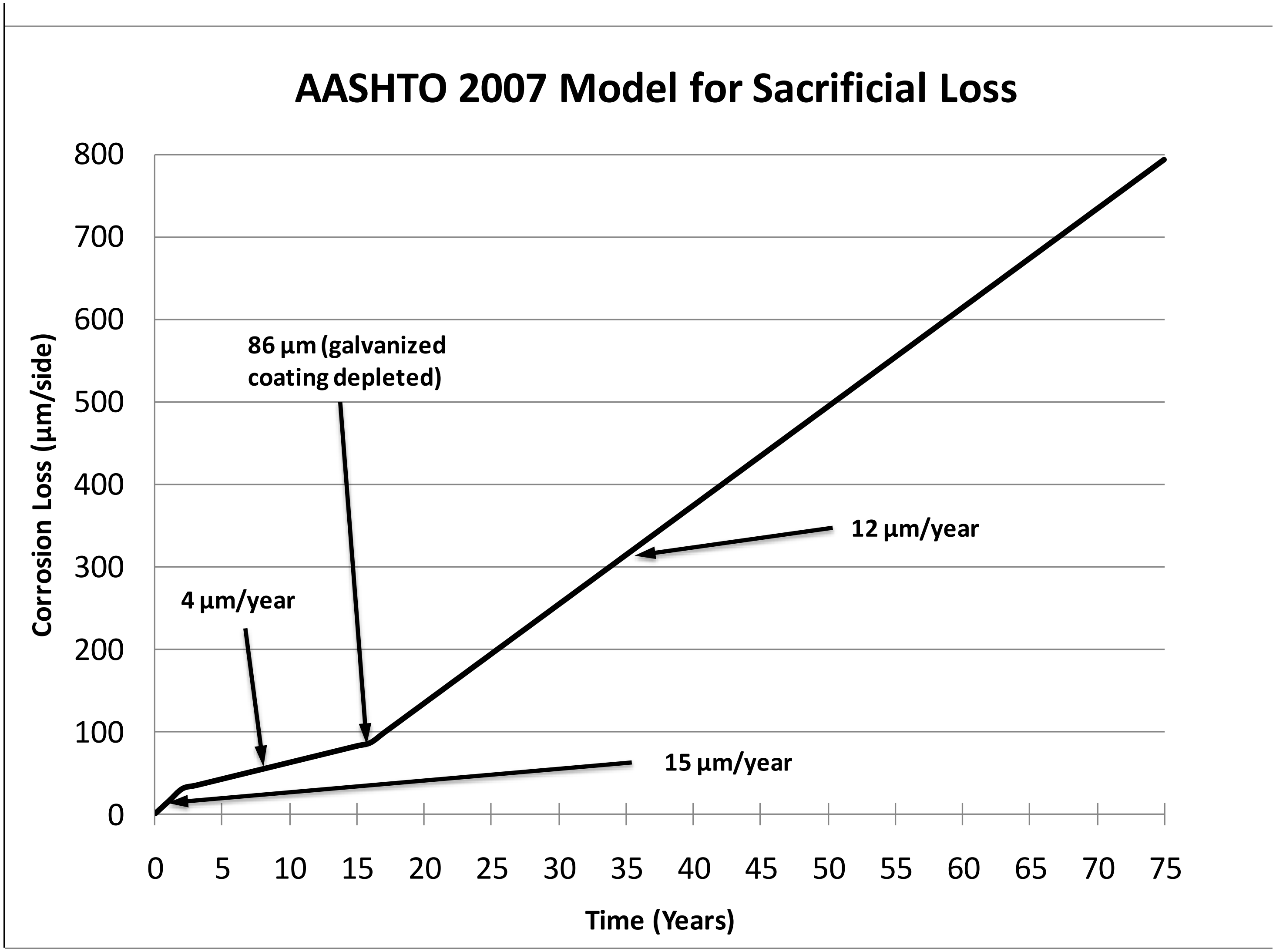

Figure 4. AASHTO Sacrificial Loss Model for Galvanized Steel with AASHTO Approved Backfill Specifications (AASHTO 2007)

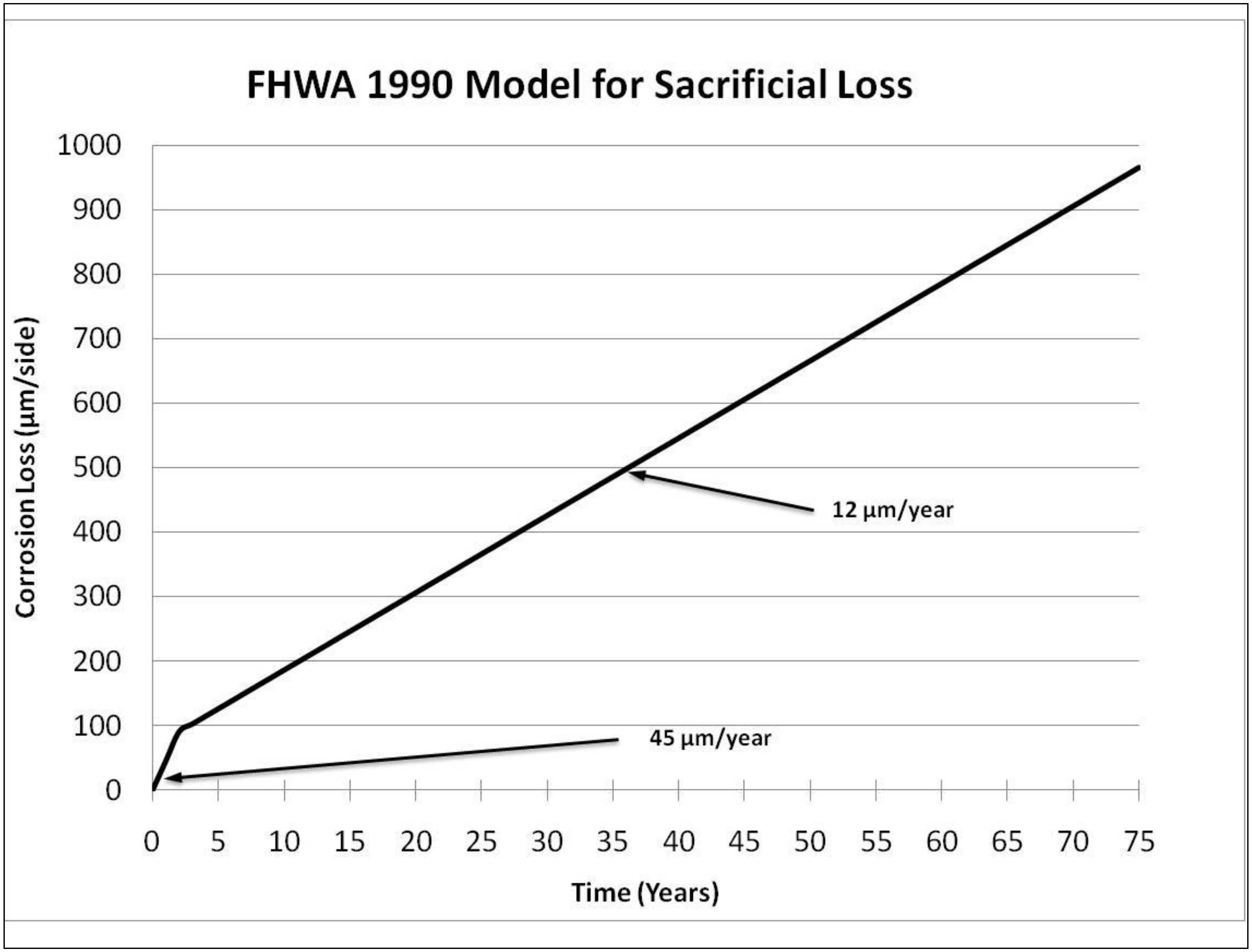

Figure 5. 1990 FHWA Sacrificial Loss Model for Black Steel with AASHTO Approved Backfill Specifications (Elias 1990)

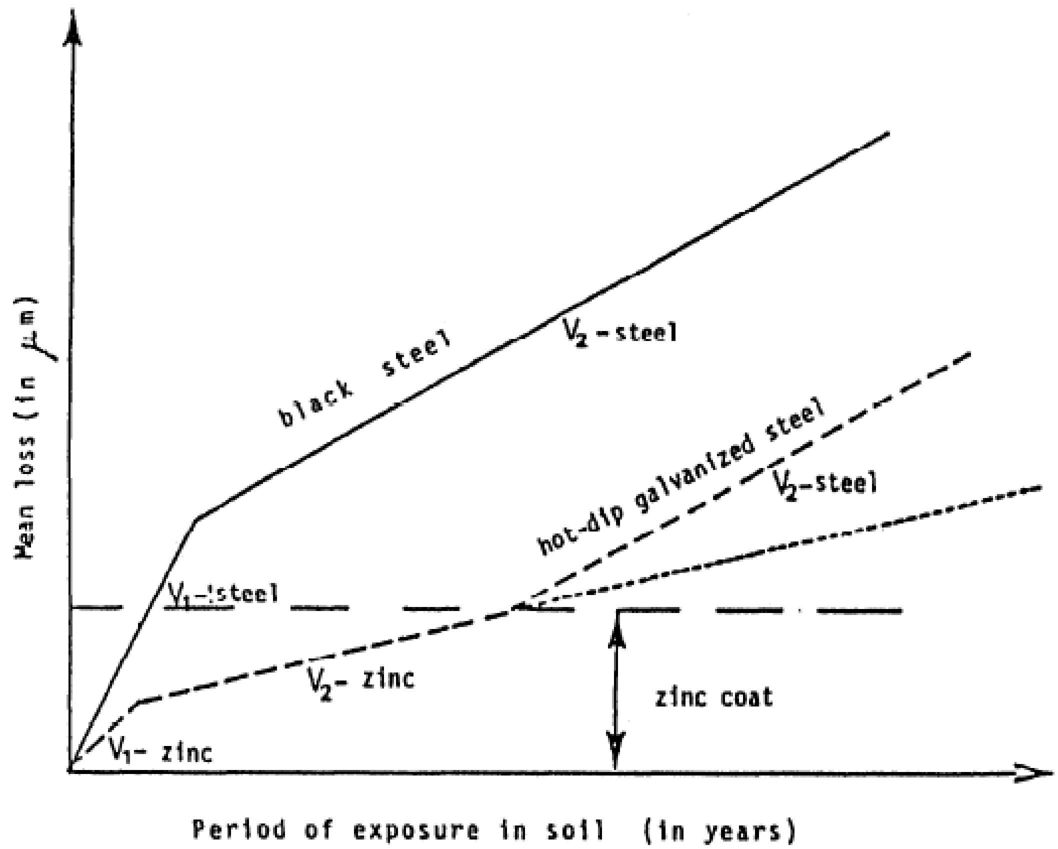

Figure 6. Idealized Corrosion Morphology with and without Zinc Coating (Elias 1990)

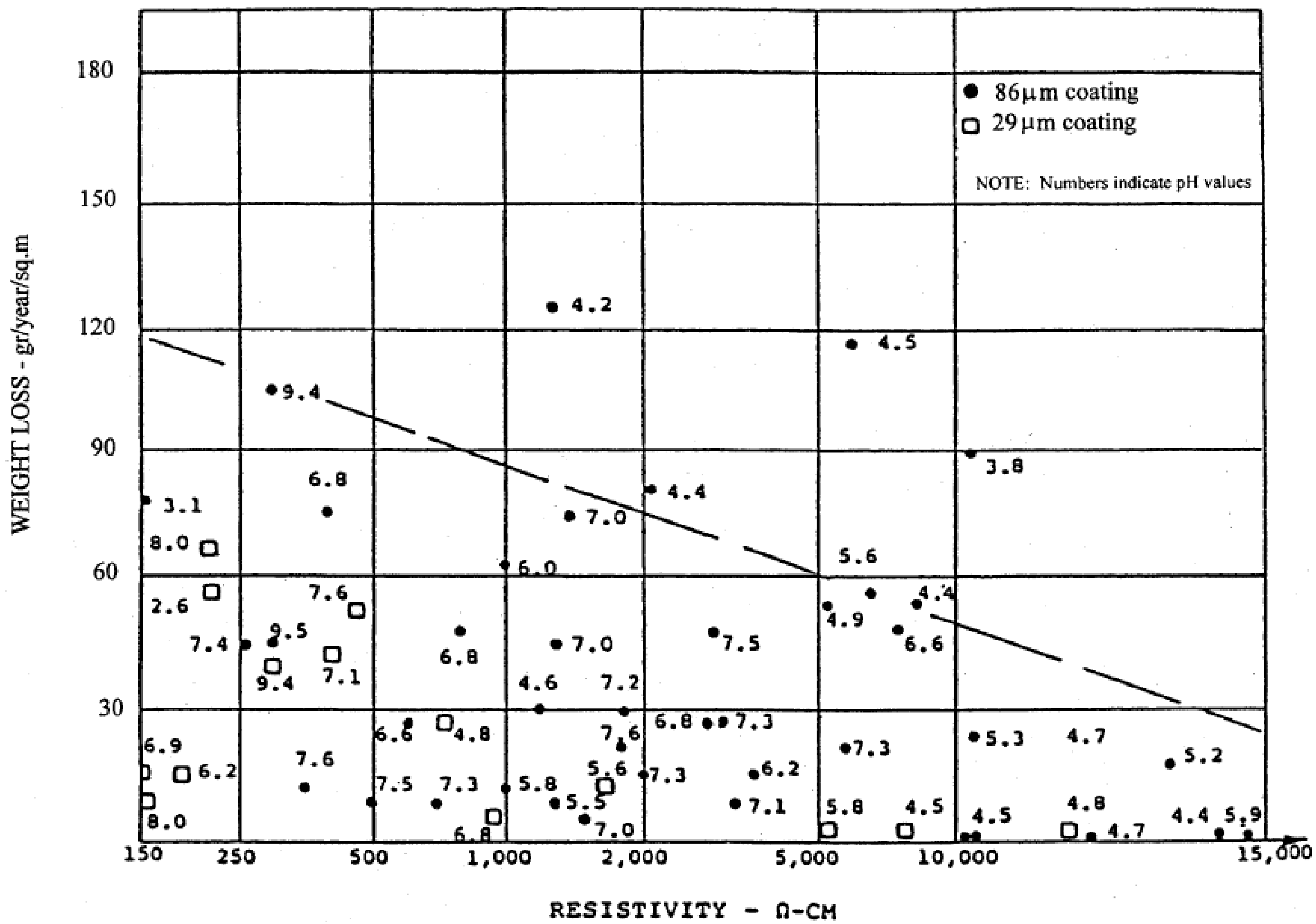

Figure 7. Metal Loss as a Function of Resistivity for Galvanized Steel (Elias 2000)

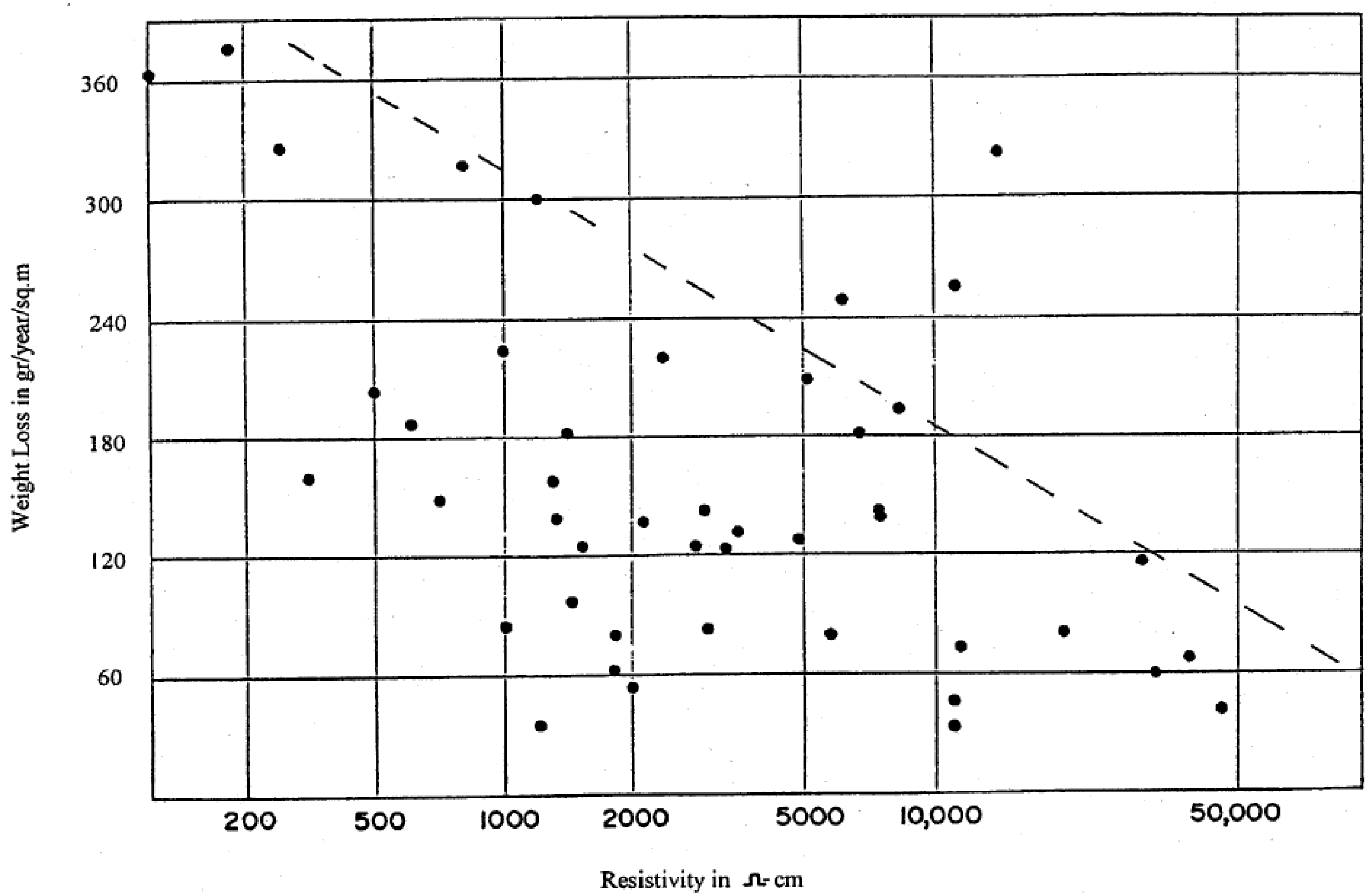

Figure 8. Metal Loss as a Function of Resistivity for Black Steel (Elias 2000)

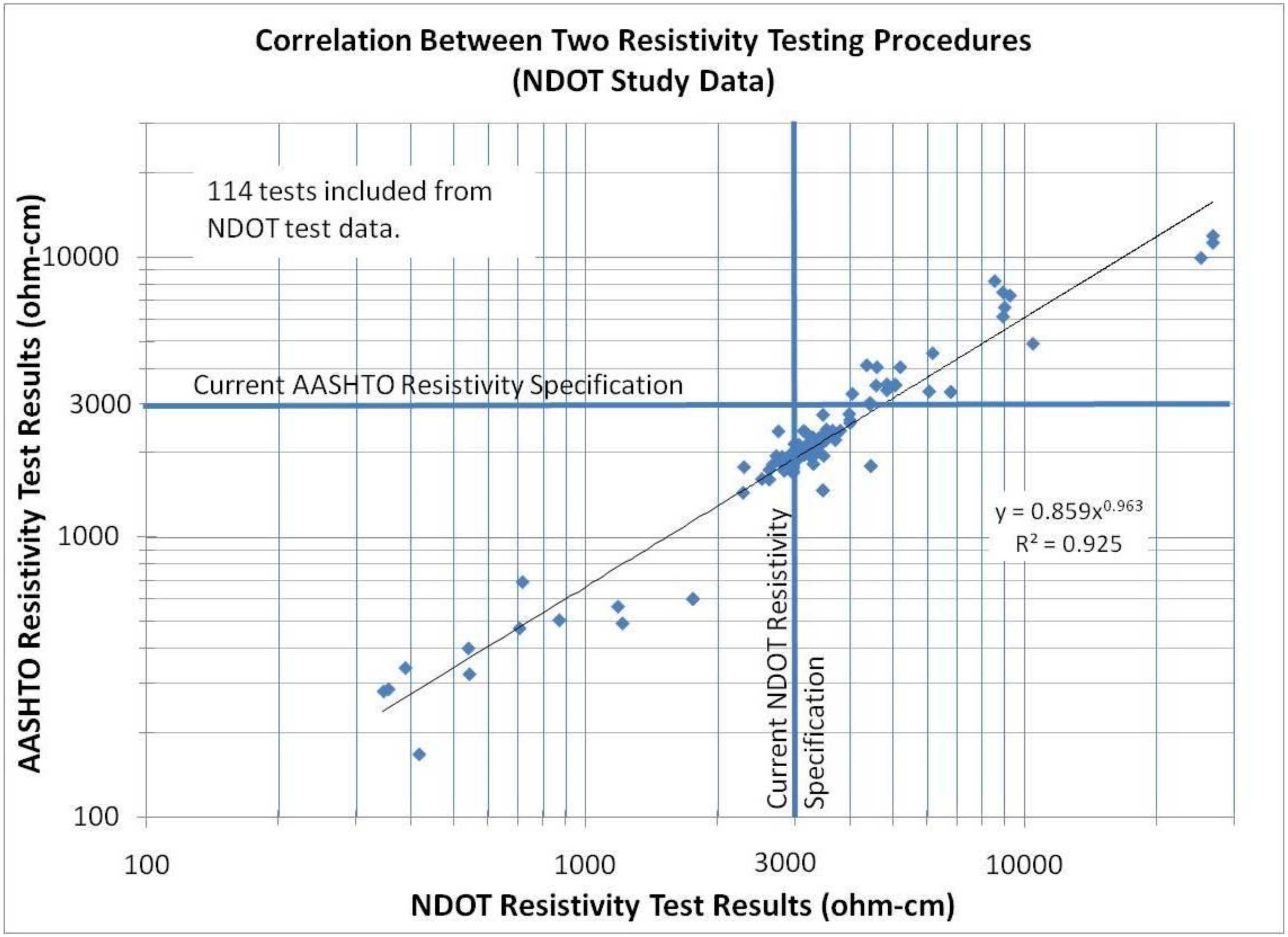

Figure 9. Correlation Relationship Between Nevada T235B and AASHTO T-288 Soil Resistivity Test Methods

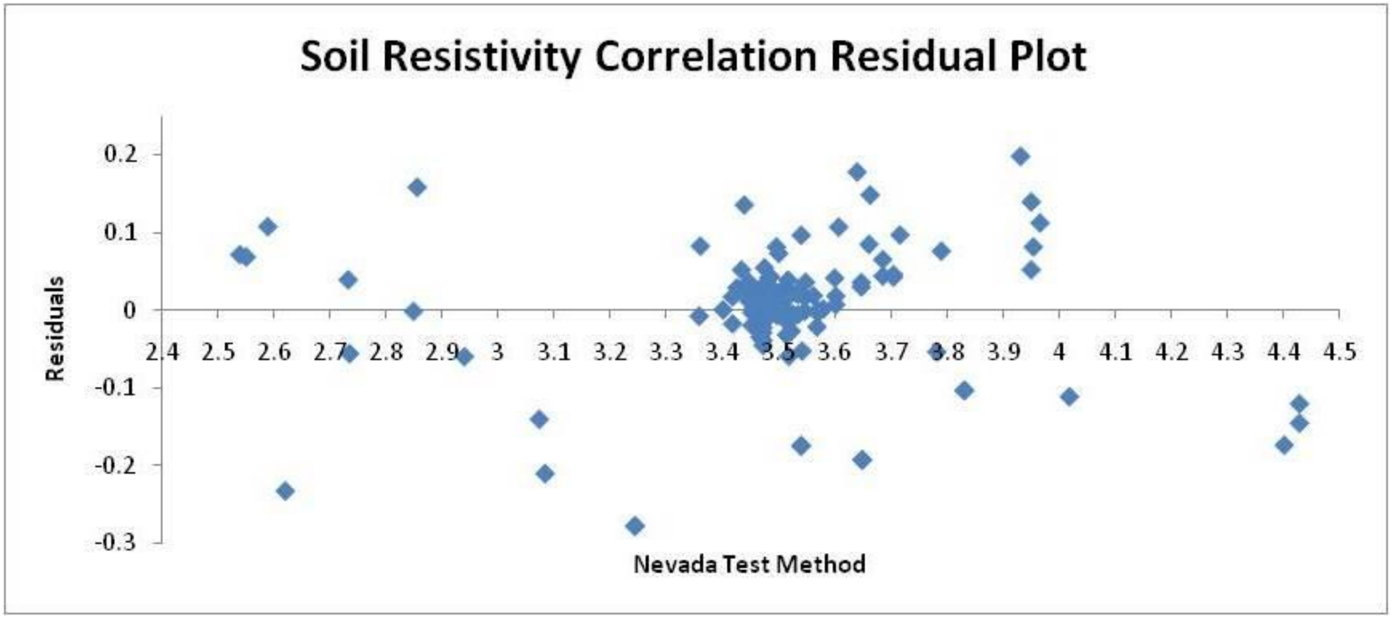

Figure 10. Correlation Relationship Residuals for the Nevada T235B and AASHTO T-288 Soil Resistivity Test Methods

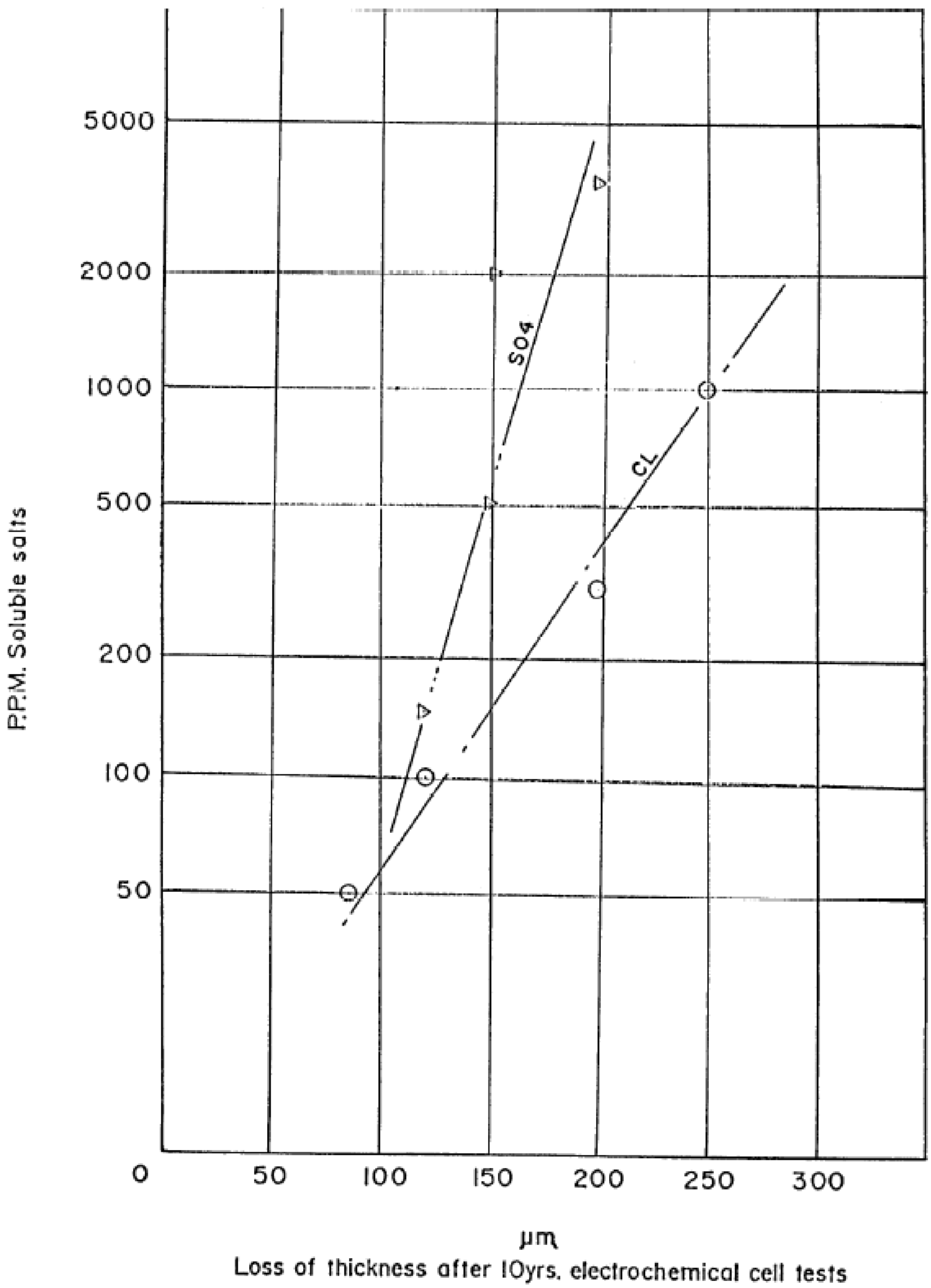

Figure 11. Soluble Salts vs. Metal Loss After 10 Years (Elias 1990)

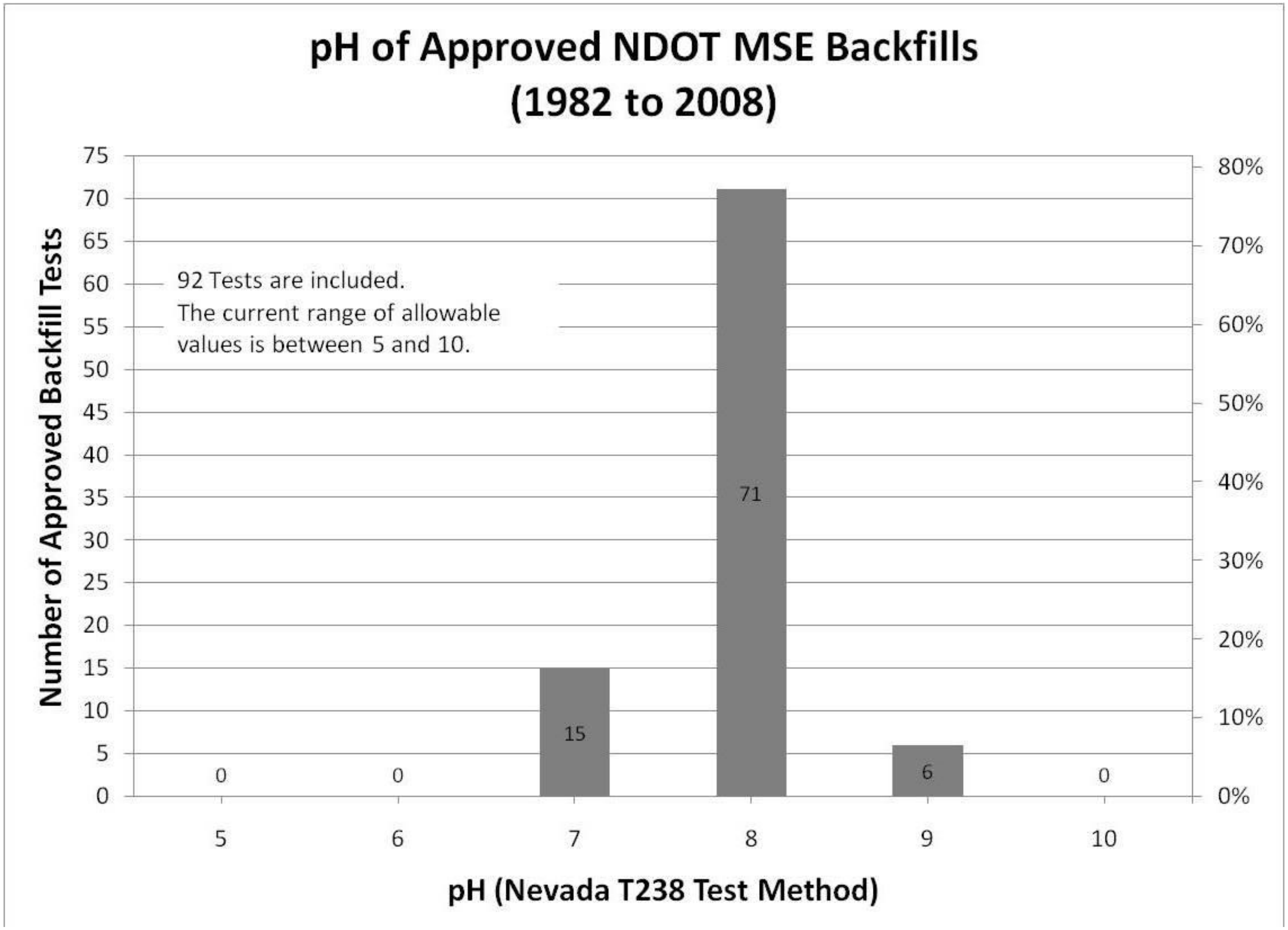

Figure 12. Distribution of pH Measurements for NDOT Approved Backfill

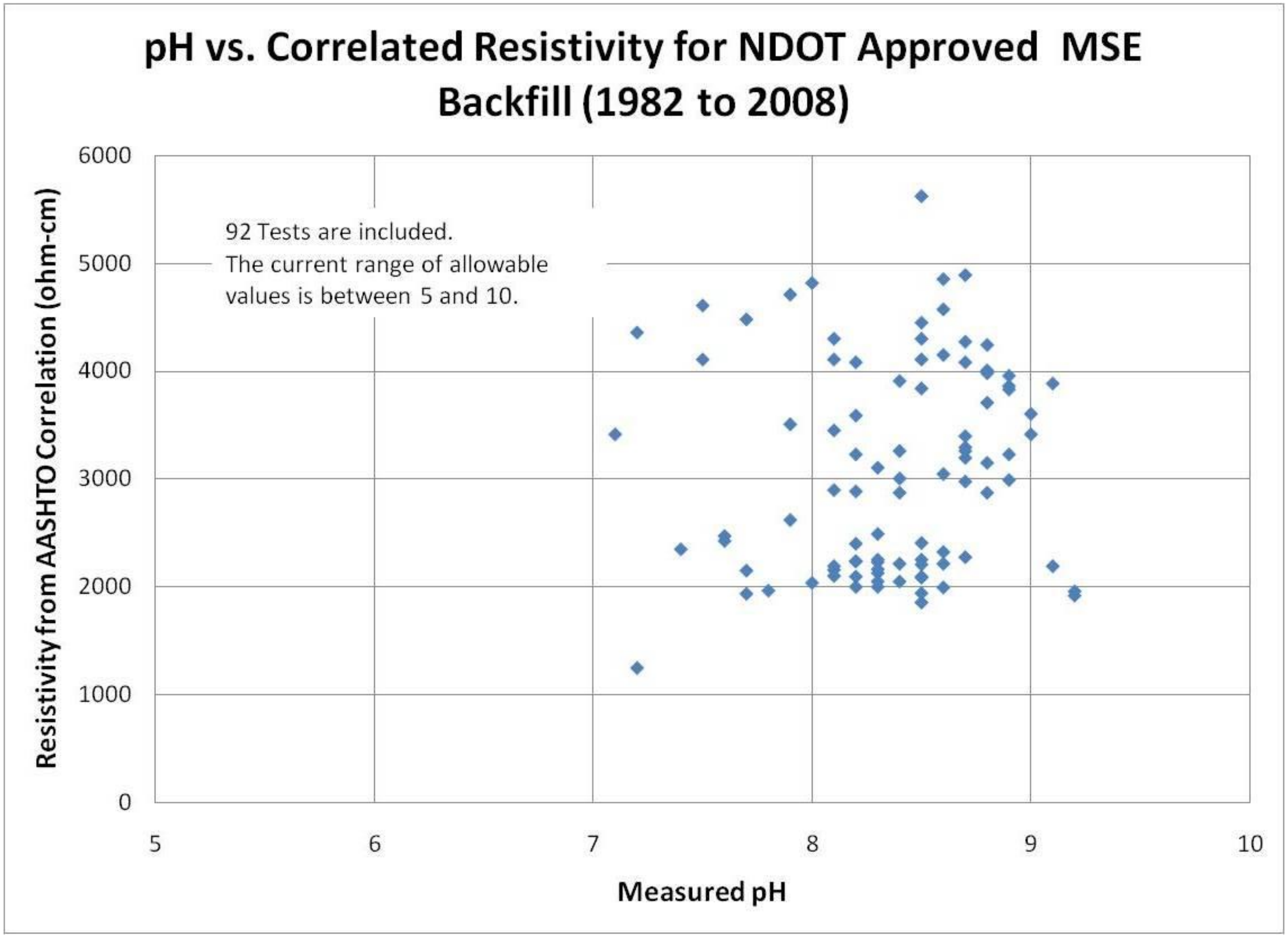

Figure 13. pH vs. Resistivity Measurements for NDOT Approved Backfill

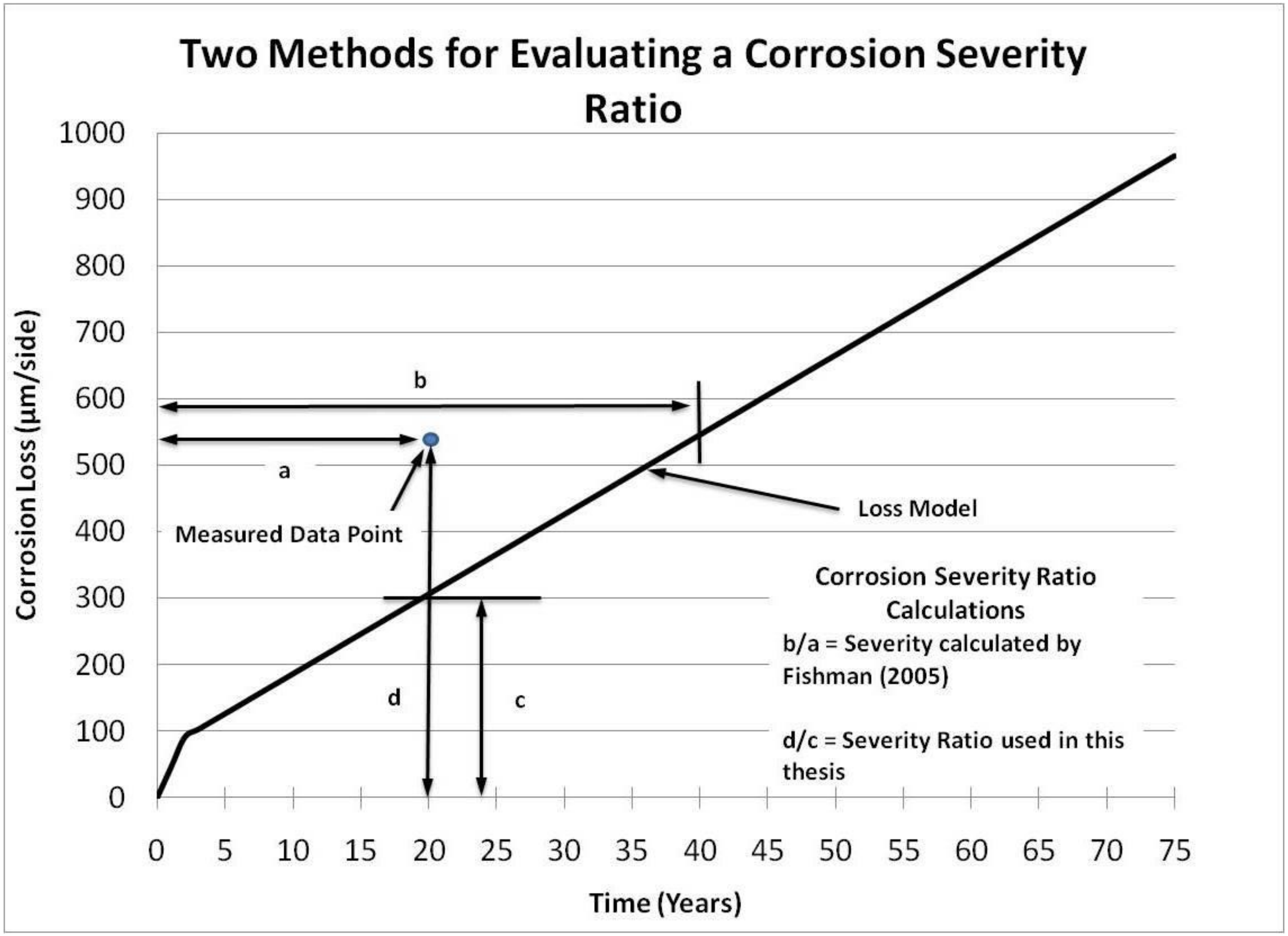

Figure 14. Methods used to Calculate and Evaluate the Corrosion Severity Ratio

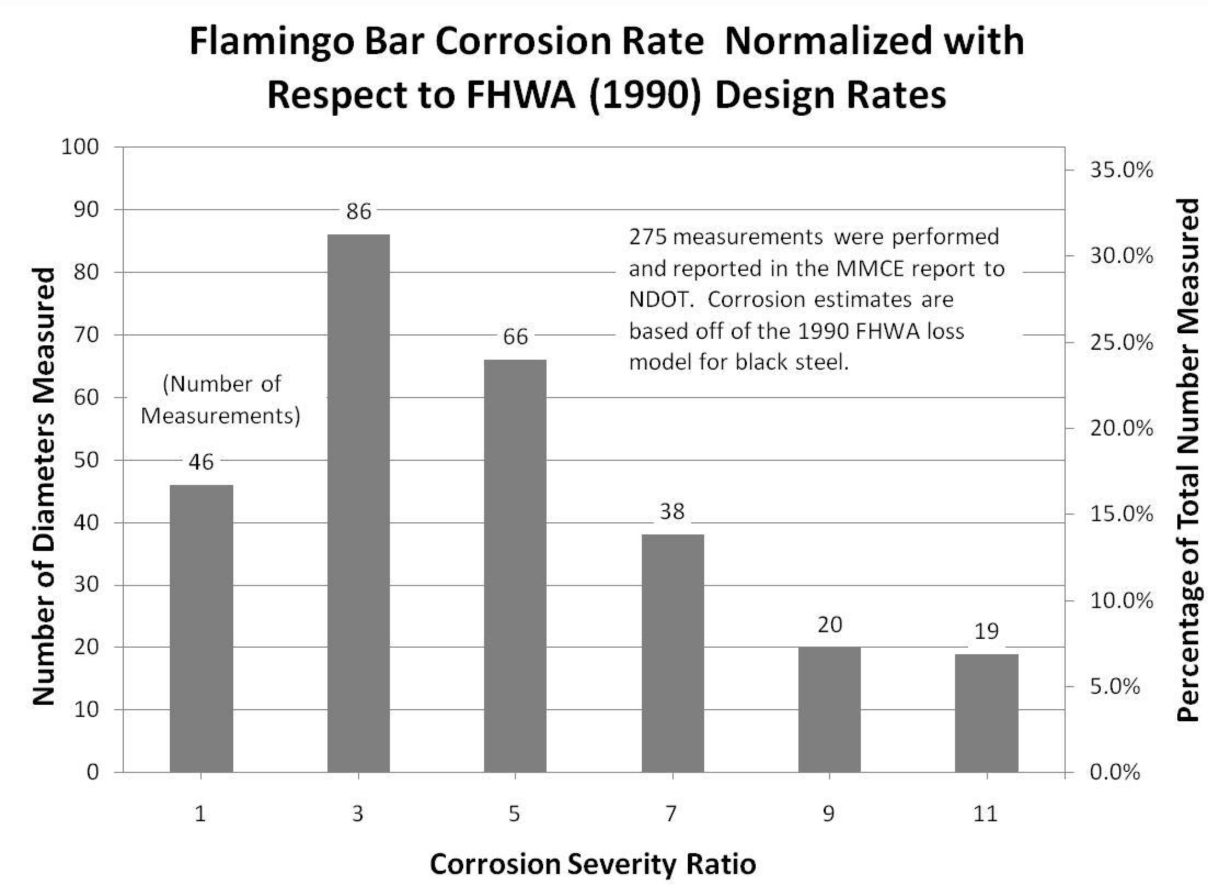

Figure 15. Distribution of Corrosion Rates Normalized with Respect to FHWA (1990) Design Rates for Flamingo Walls #2 and #3

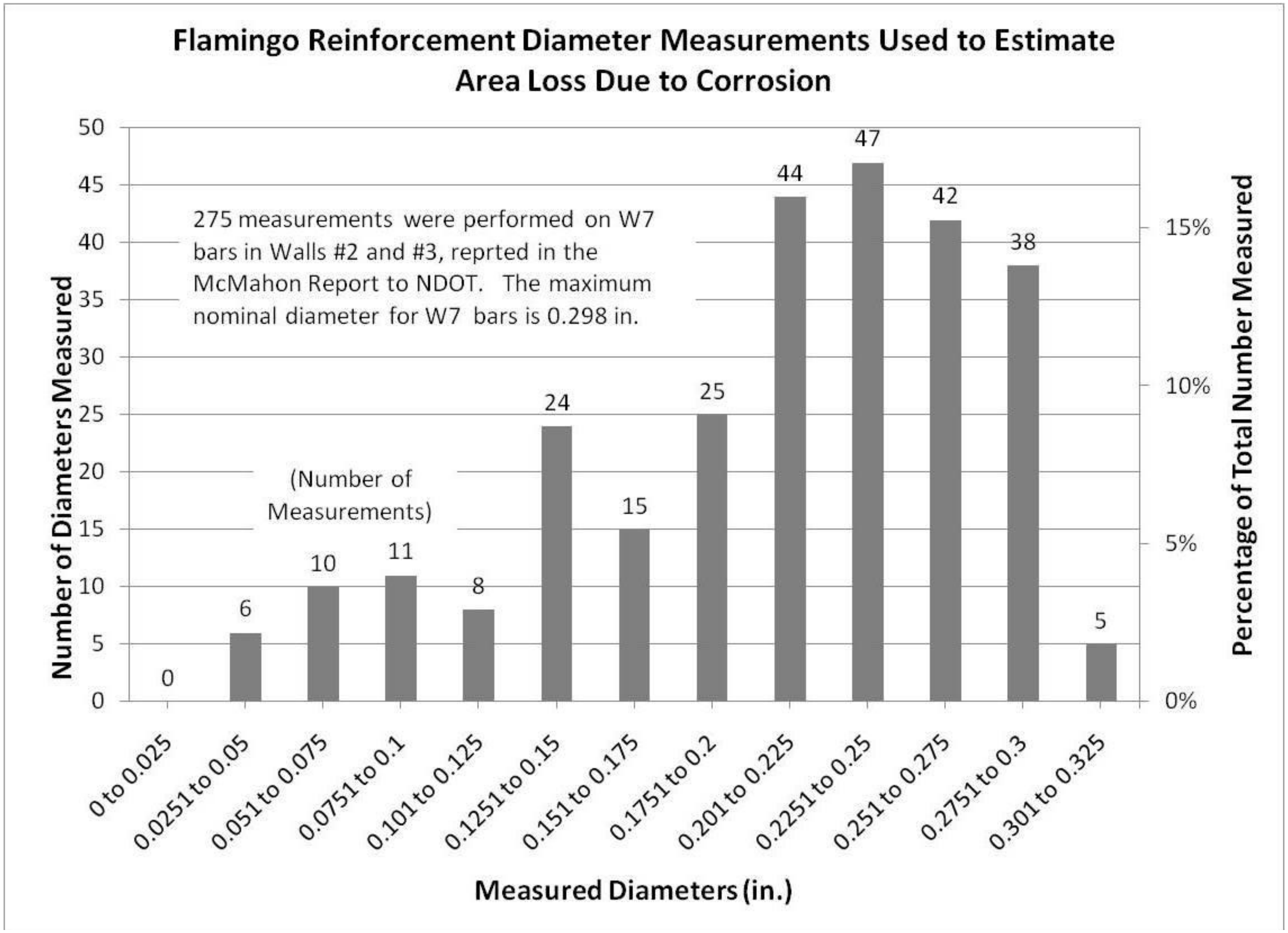

Figure 16. Distribution of Diameter Measurements for Flamingo Walls #2 and #3

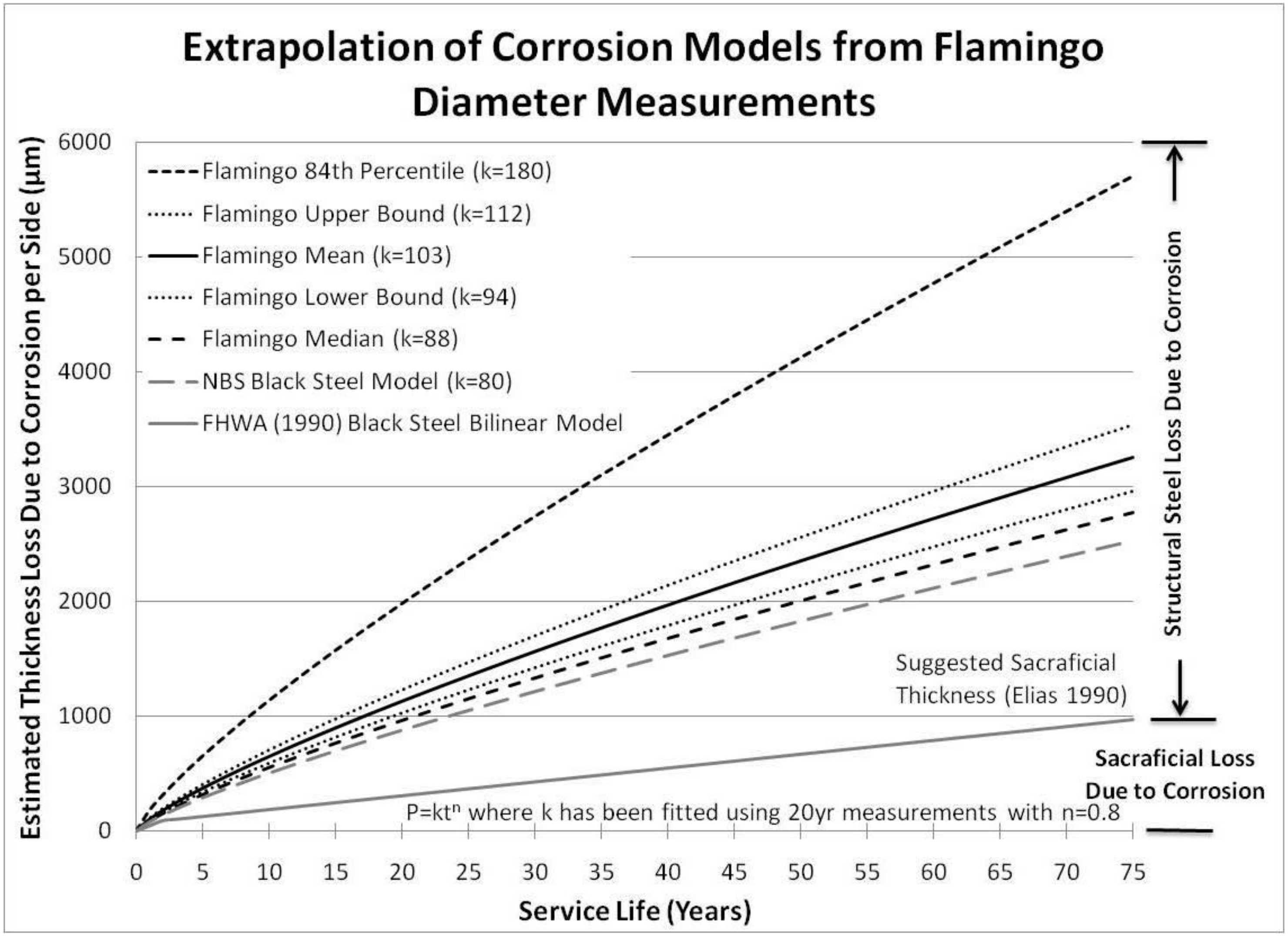

Figure 17. Extrapolation of Corrosion Loss Models from Flamingo Diameter Measurements

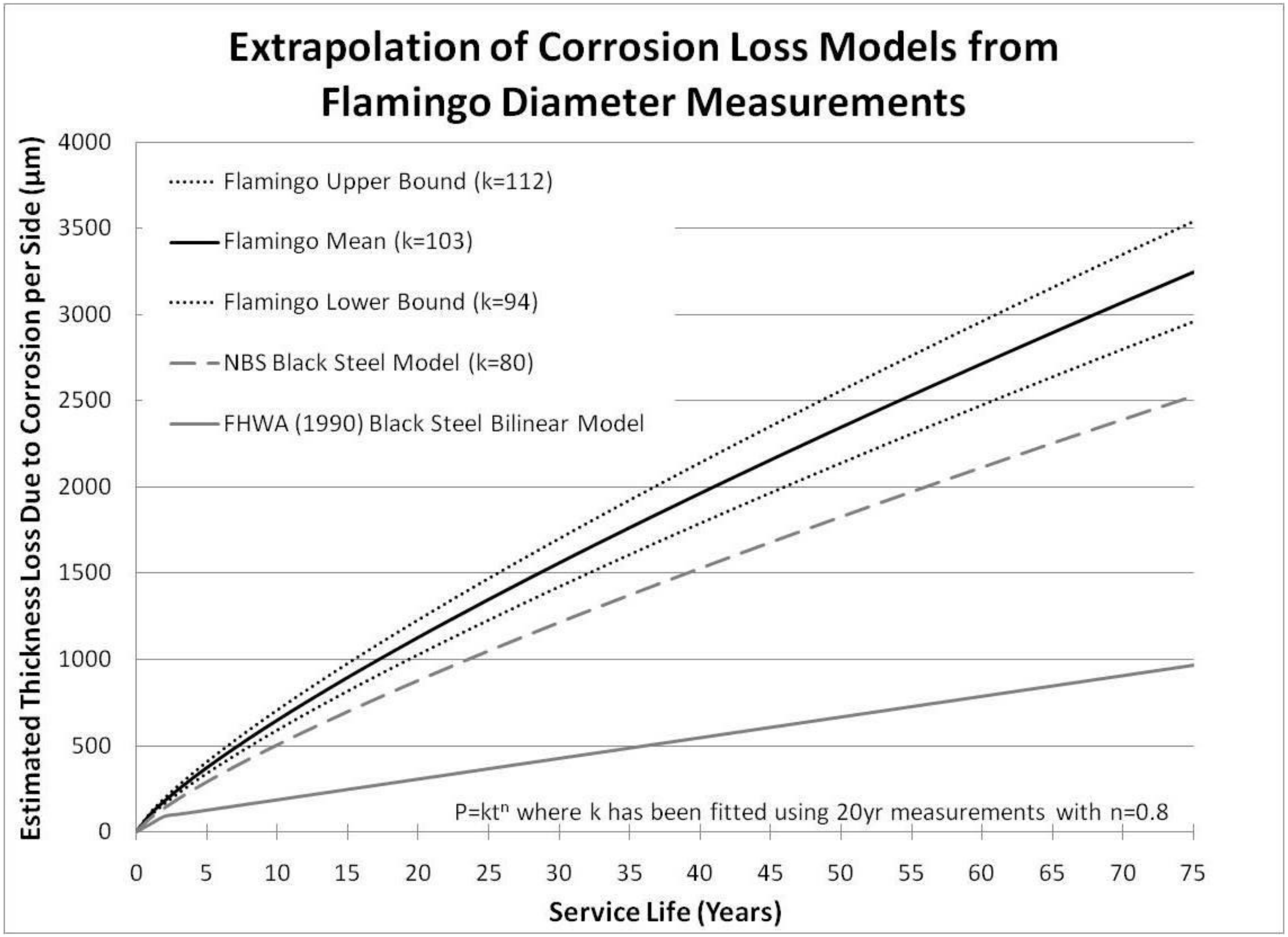

Figure 18. Extrapolation of Corrosion Loss Models from Flamingo Diameter Measurements (Reproduced from Figure 17 for Clarity)

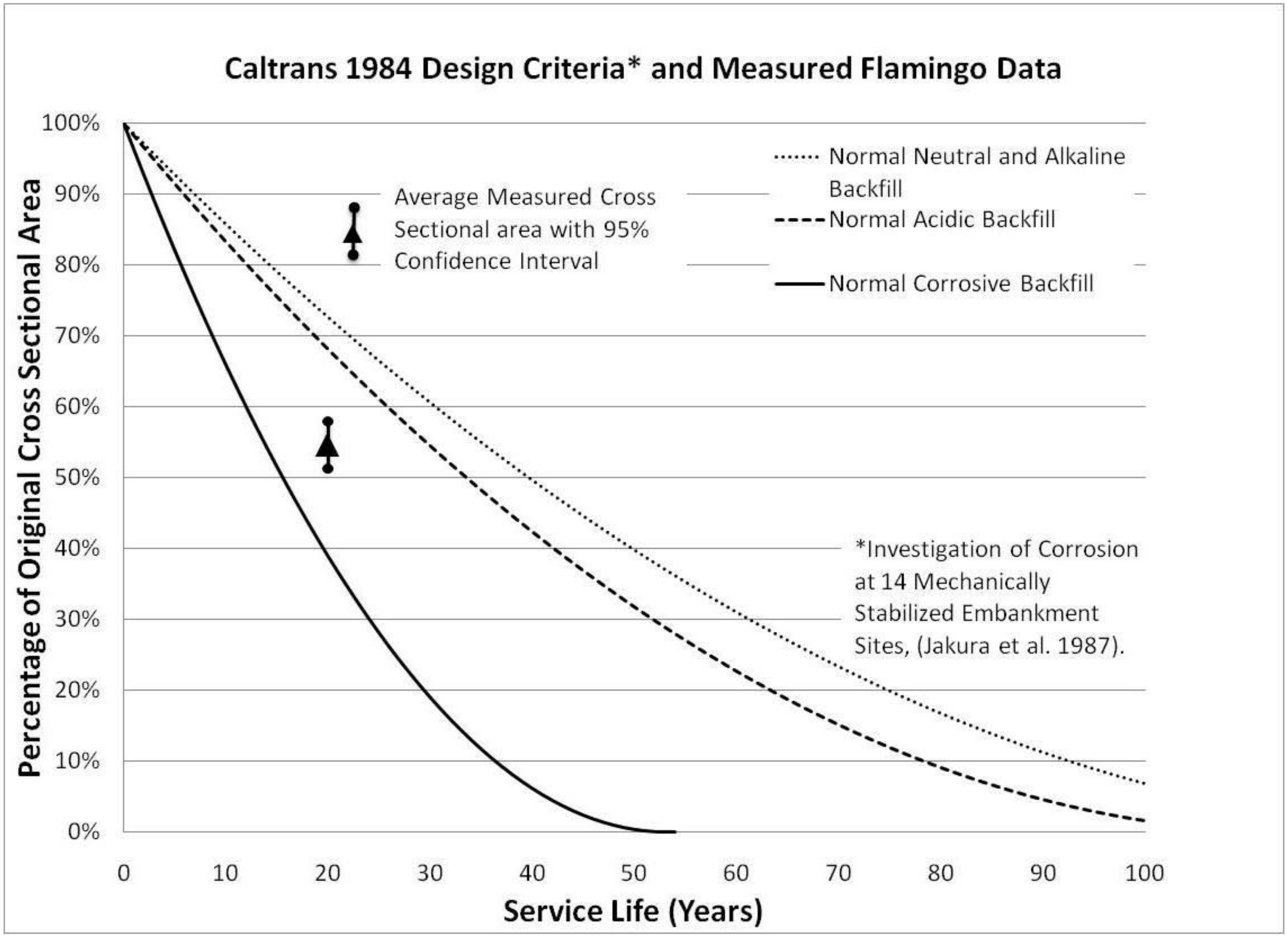

Figure 19. Potential Corrosion Loss Predicted by Flamingo Corroded Diameter Measurements Compared to Caltrans (1984) Model

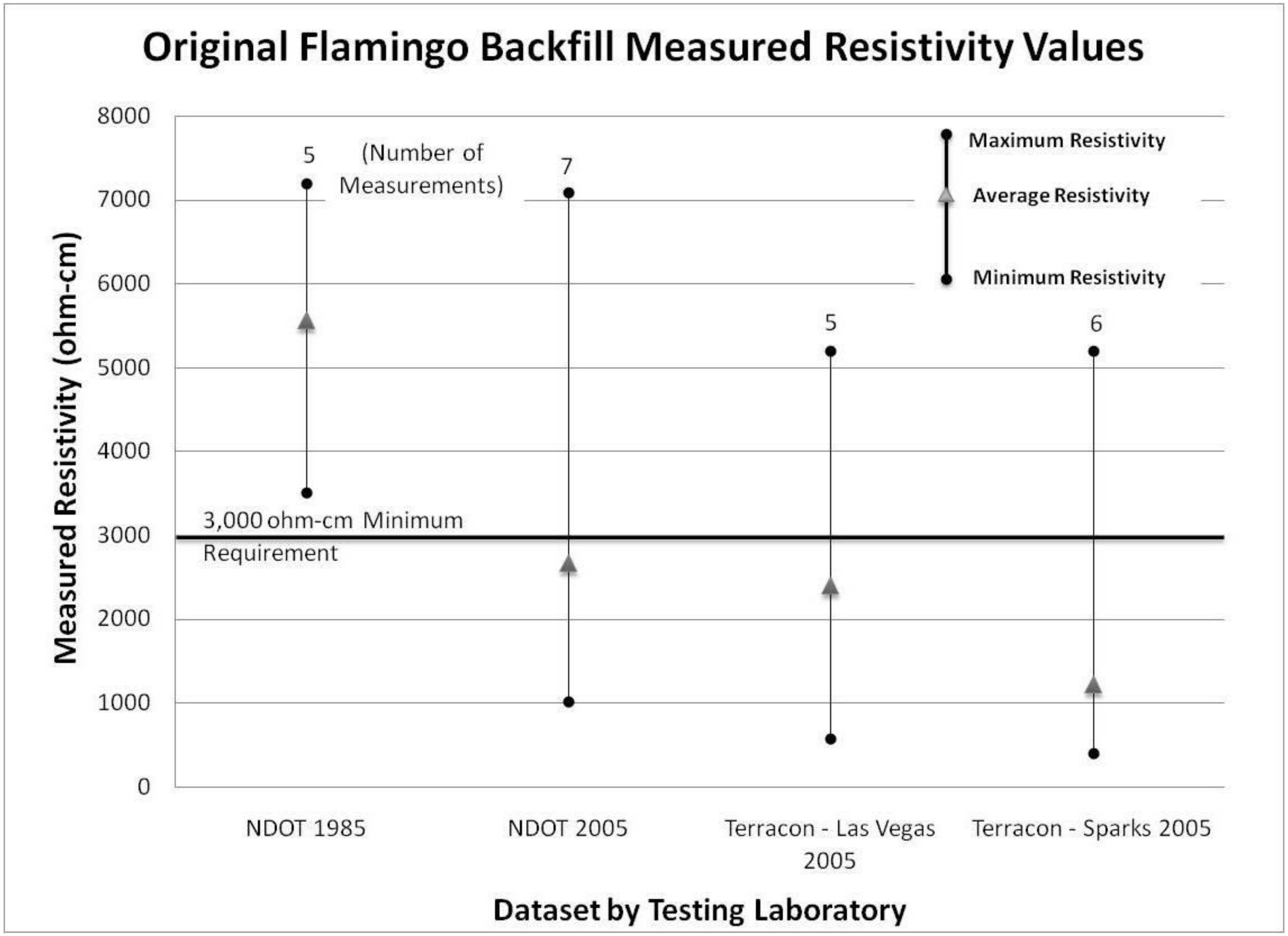

Figure 20. Ranges of Original Flamingo Measured Resistivity Values (by Laboratory)

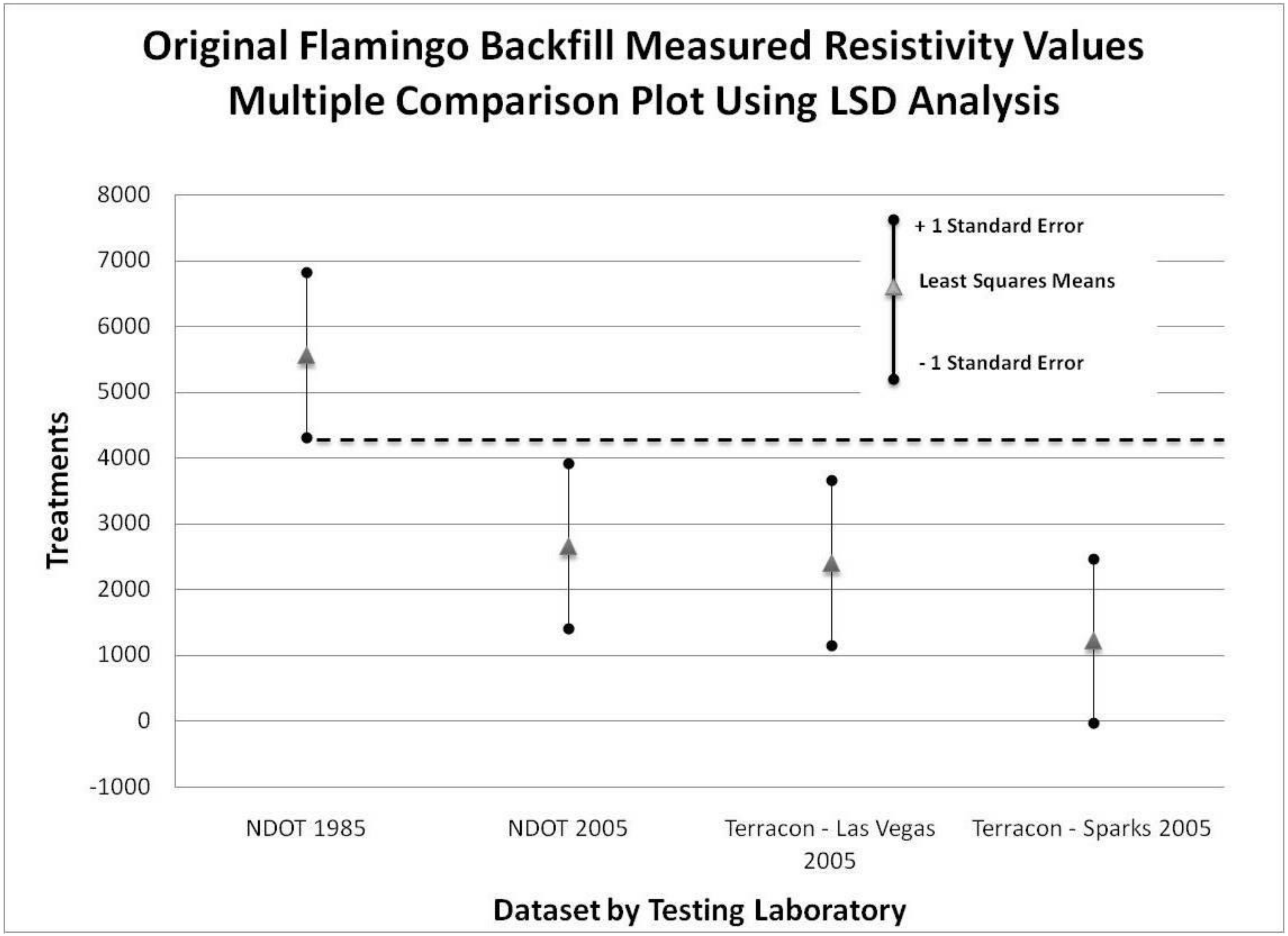

Figure 21. Original Flamingo Measured Resistivity LSD Analysis Results

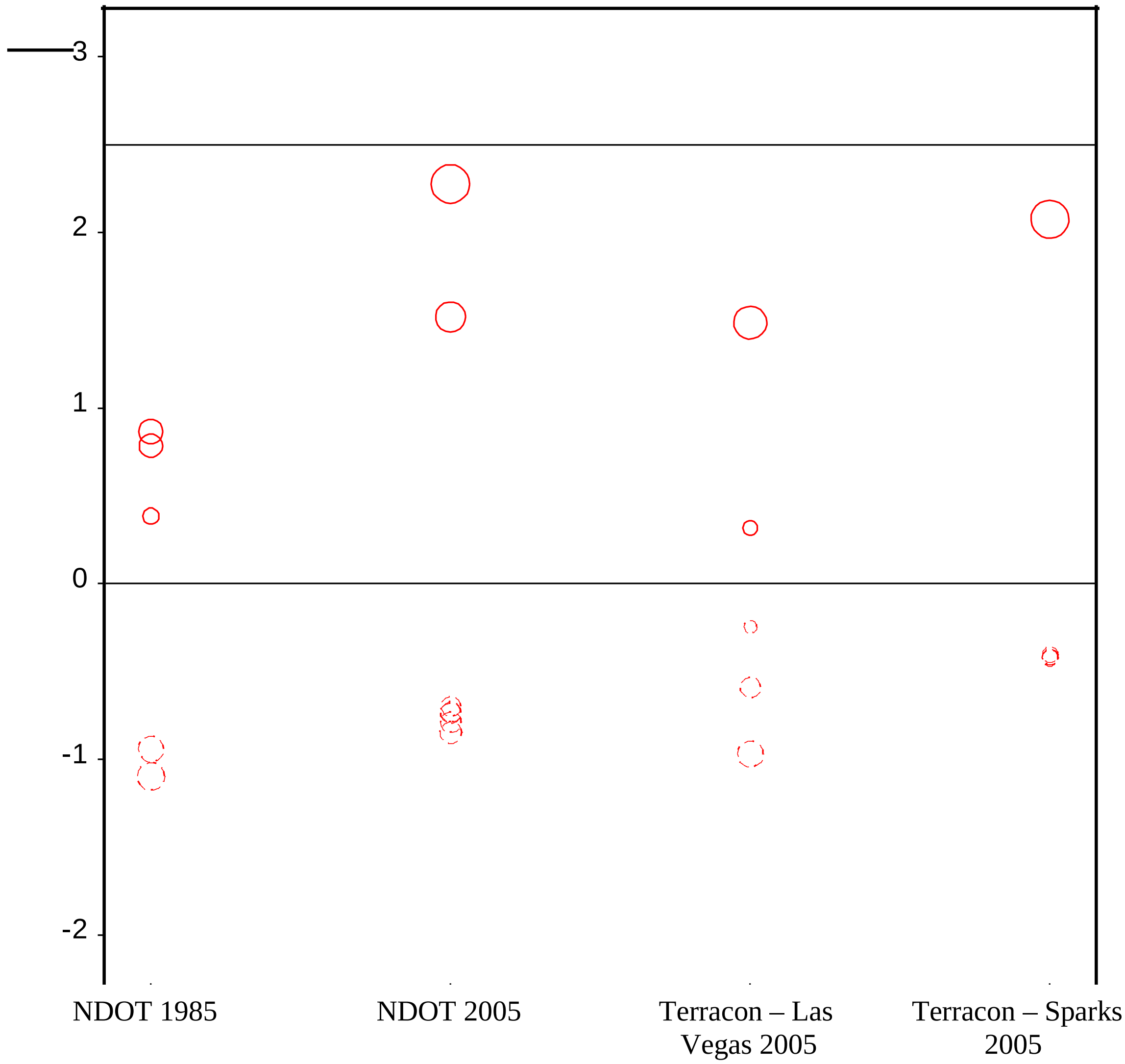

Figure 22. Original Flamingo Measured Resistivity Analysis Residuals

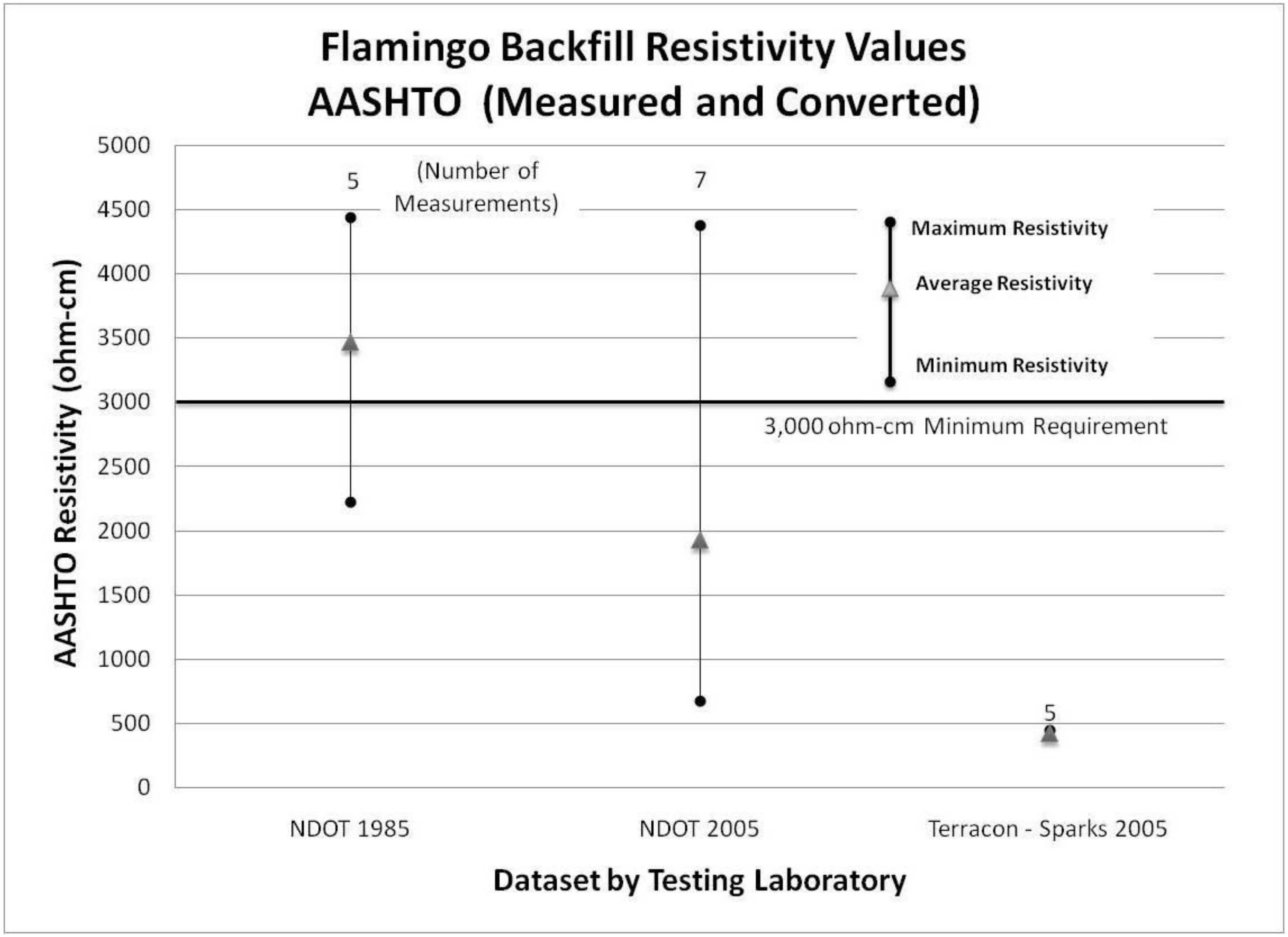

Figure 23. Ranges for Flamingo AASHTO Measured and Converted Resistivity Results

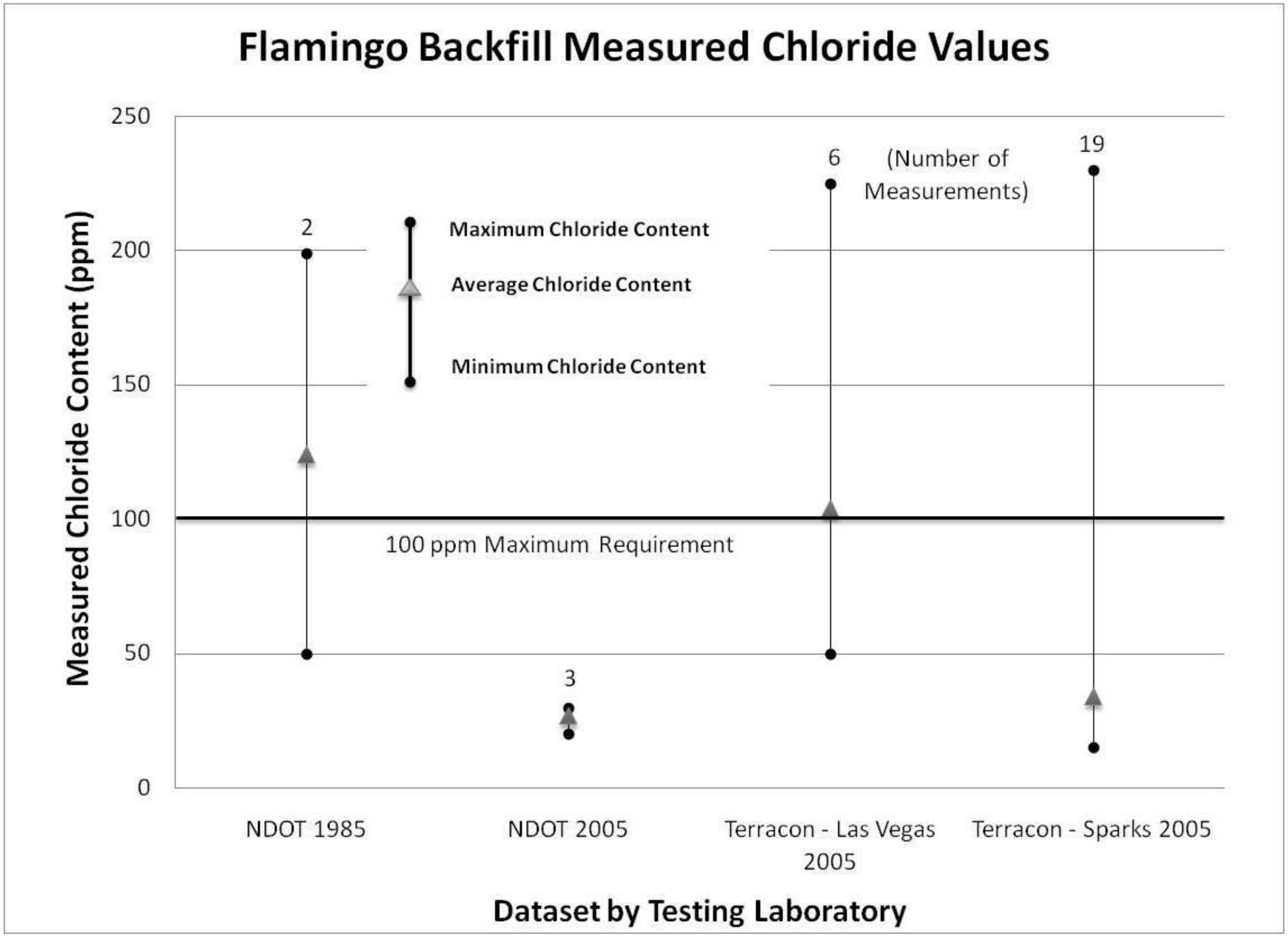

Figure 24. Ranges of Flamingo Chloride Content Results (by Laboratory)

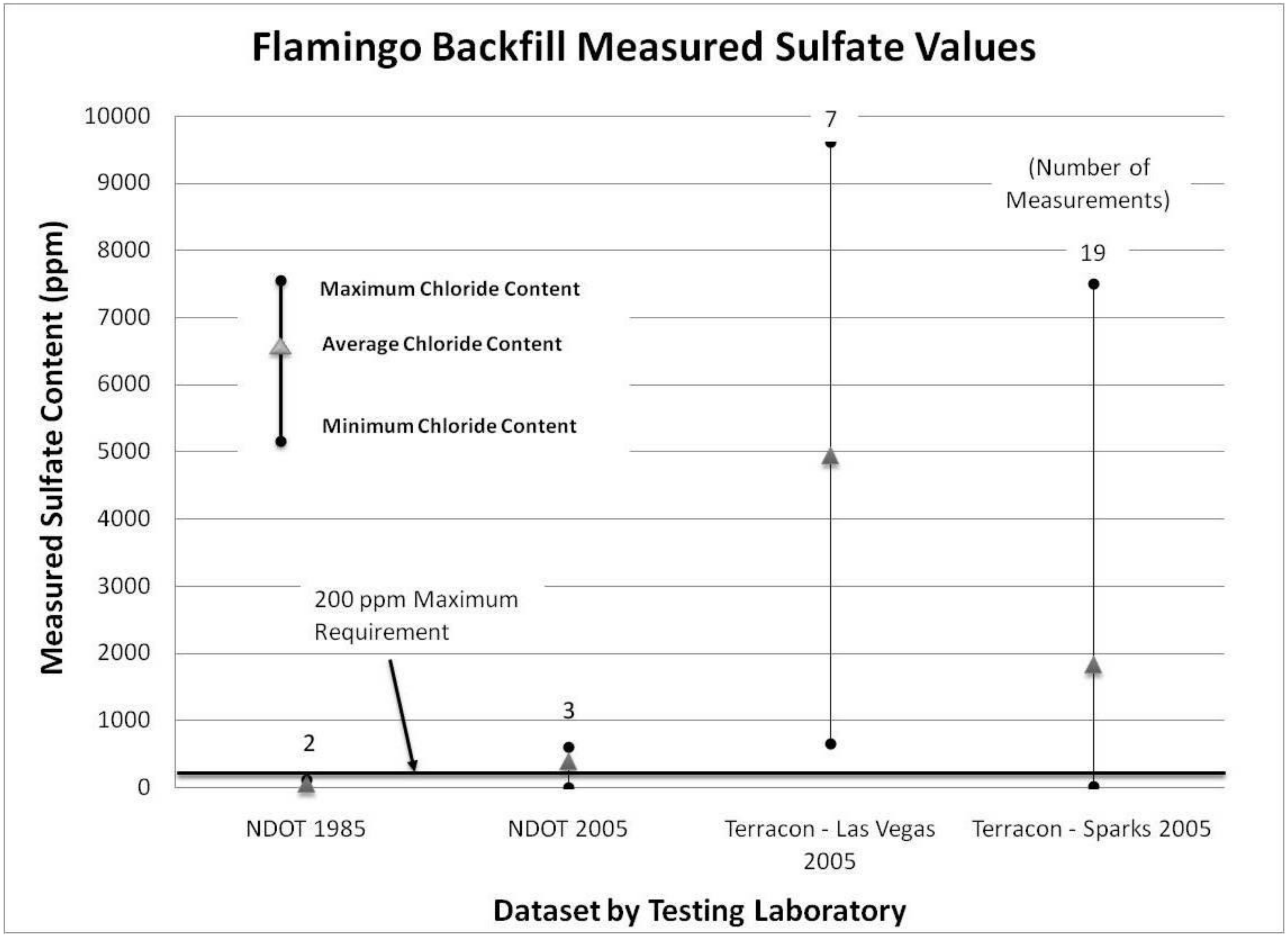

Figure 25. Ranges of Flamingo Sulfate Content Results (by Laboratory)

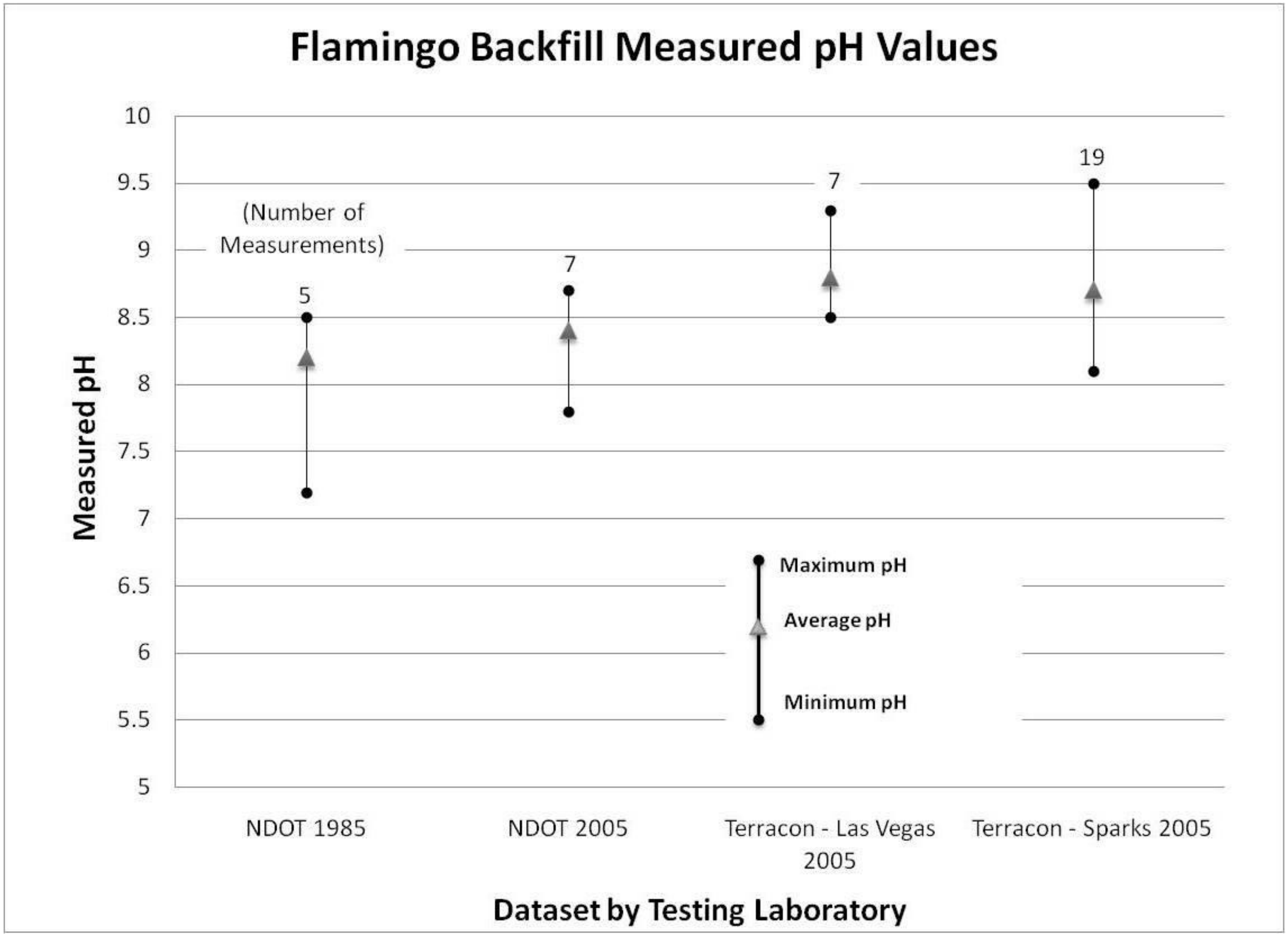

Figure 26. Ranges of Flamingo pH Results (by Laboratory)

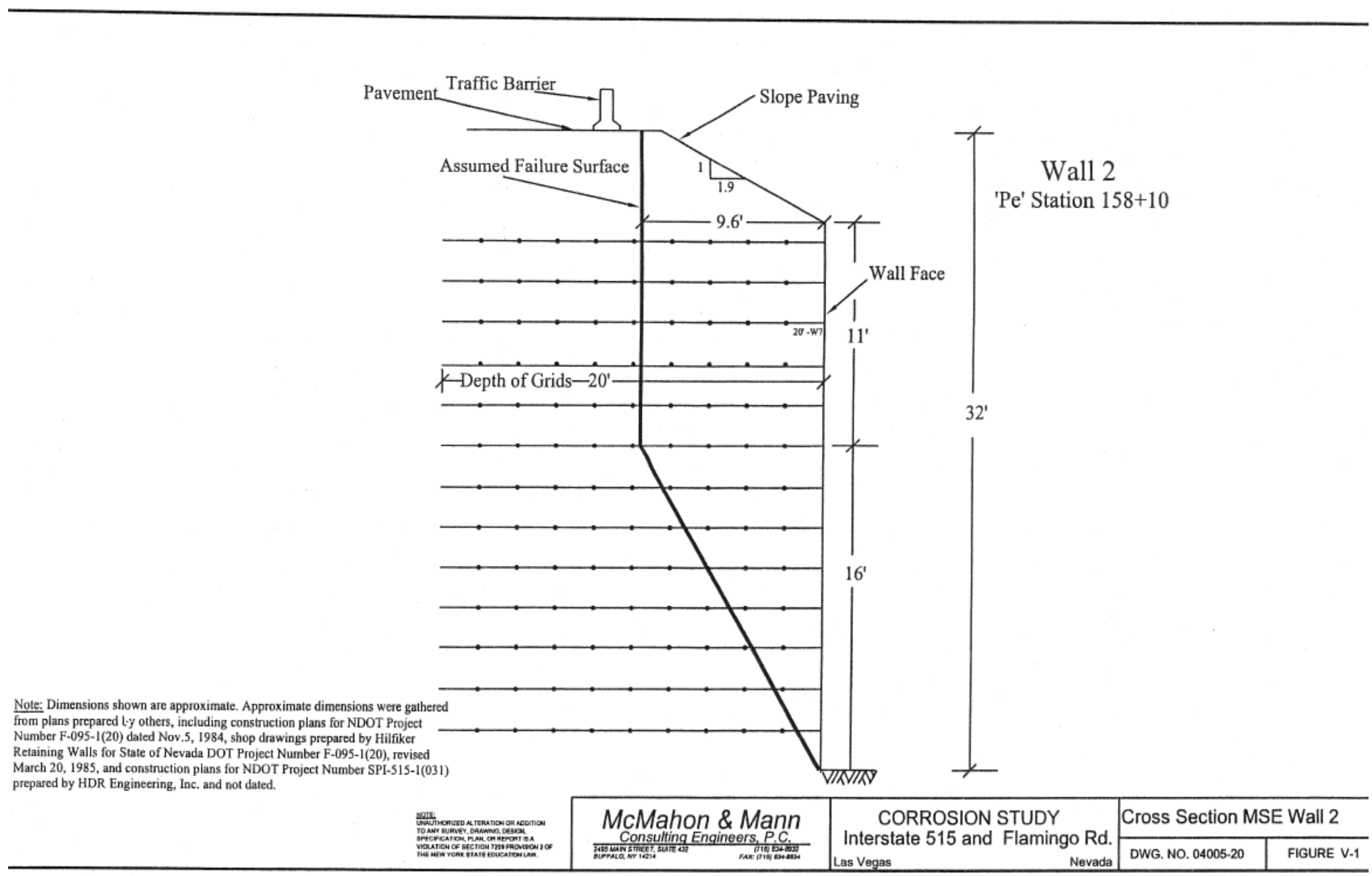

Figure 27. Flamingo Wall #2 Analysis Section (Fishman 2005)

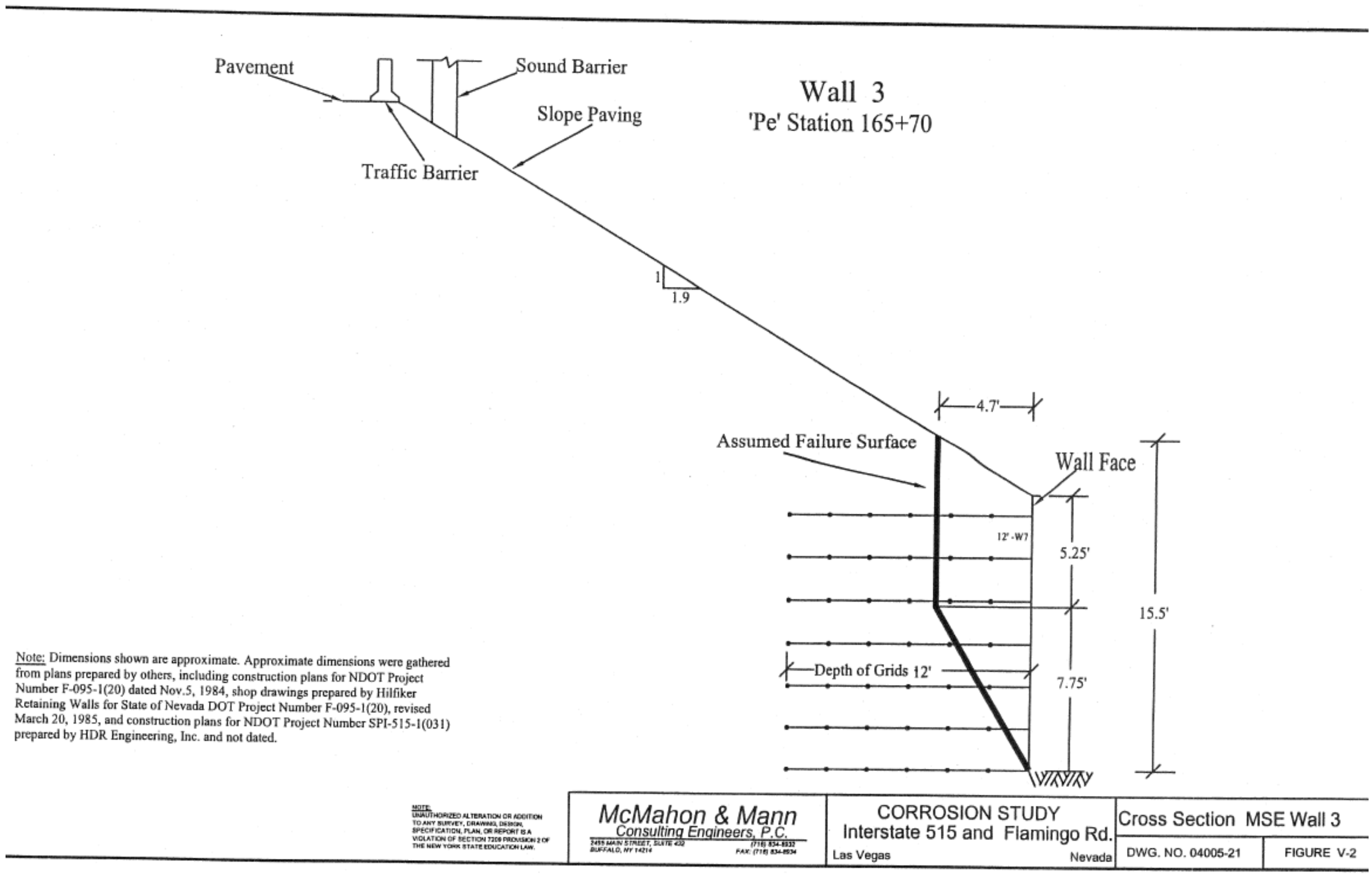

Figure 28. Flamingo Wall #3 Analysis Section (Fishman 2005)

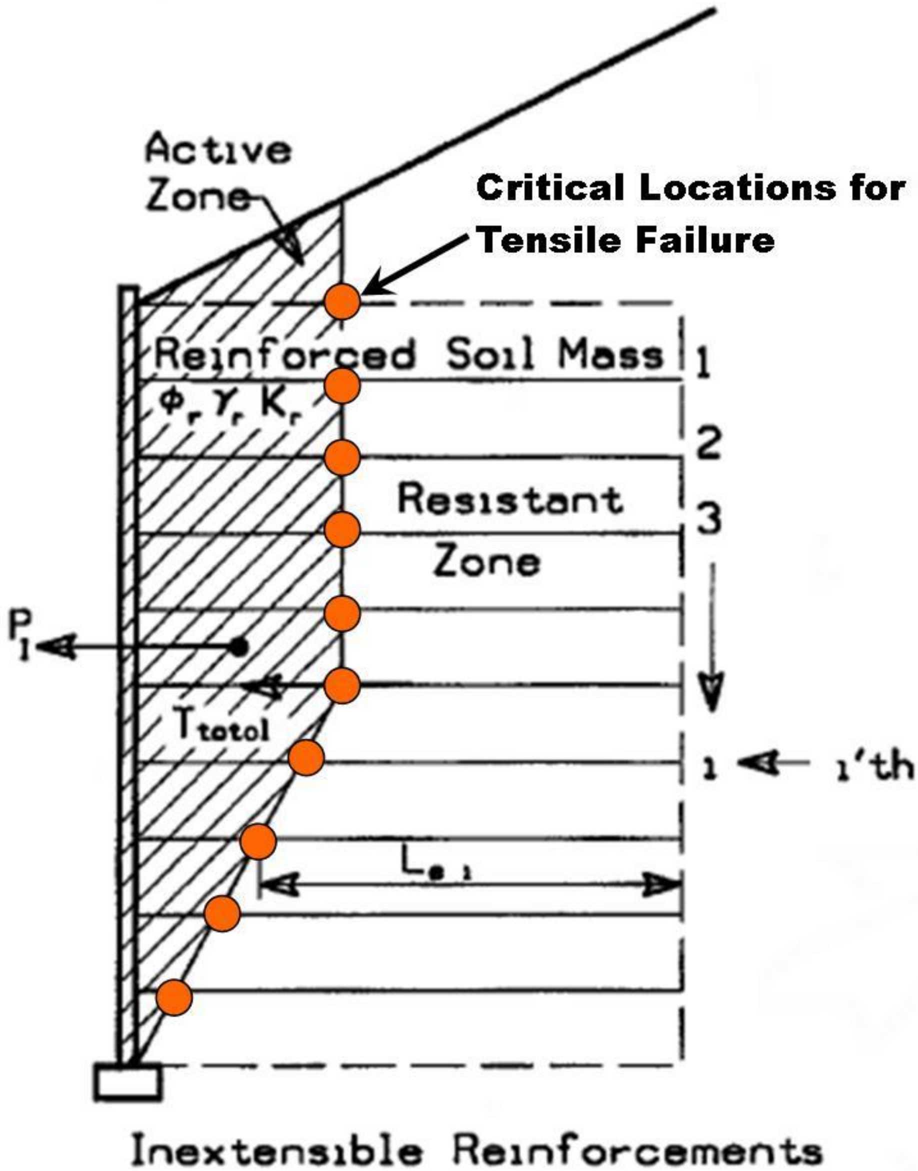

Figure 29. AASHTO Active Wedge with Critical Locations for Tensile Failure (Modified from Elias 2001)

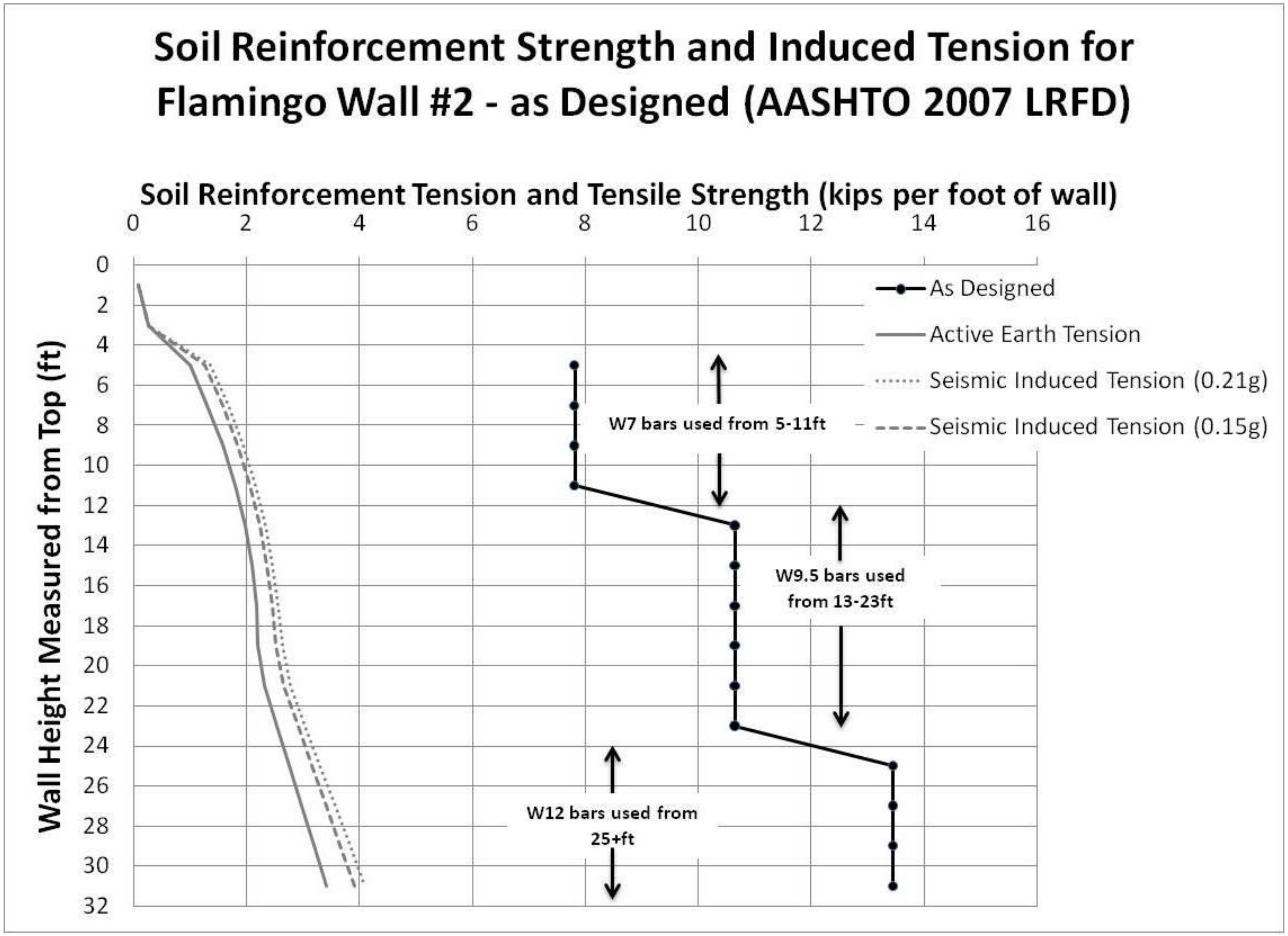

Figure 30. Soil Reinforcement Strength and Induced Tension for Flamingo Wall #2 – As Designed

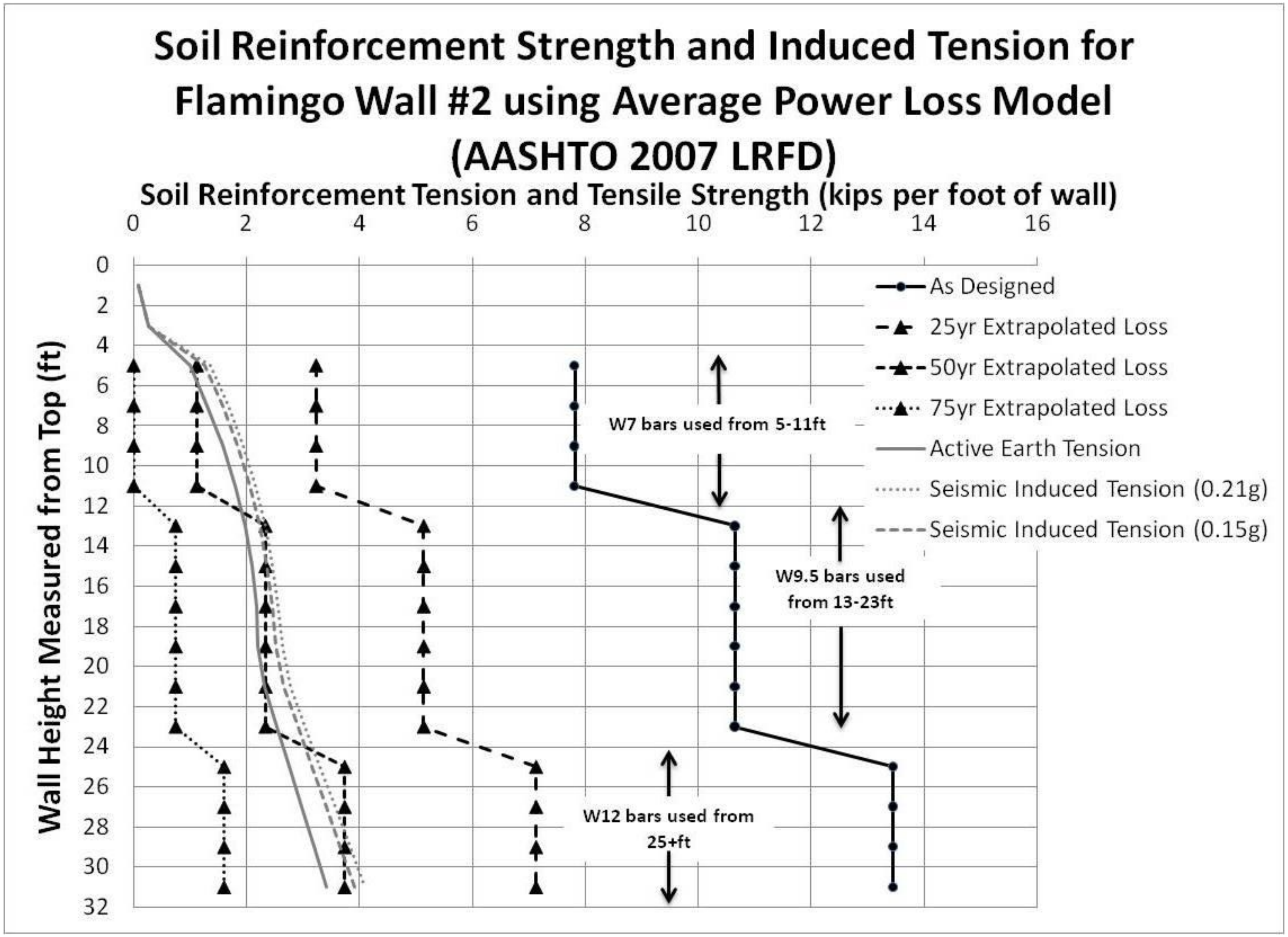

Figure 31. Soil Reinforcement Strength and Induced Tension for Flamingo Wall #2 – Average Power Loss Model

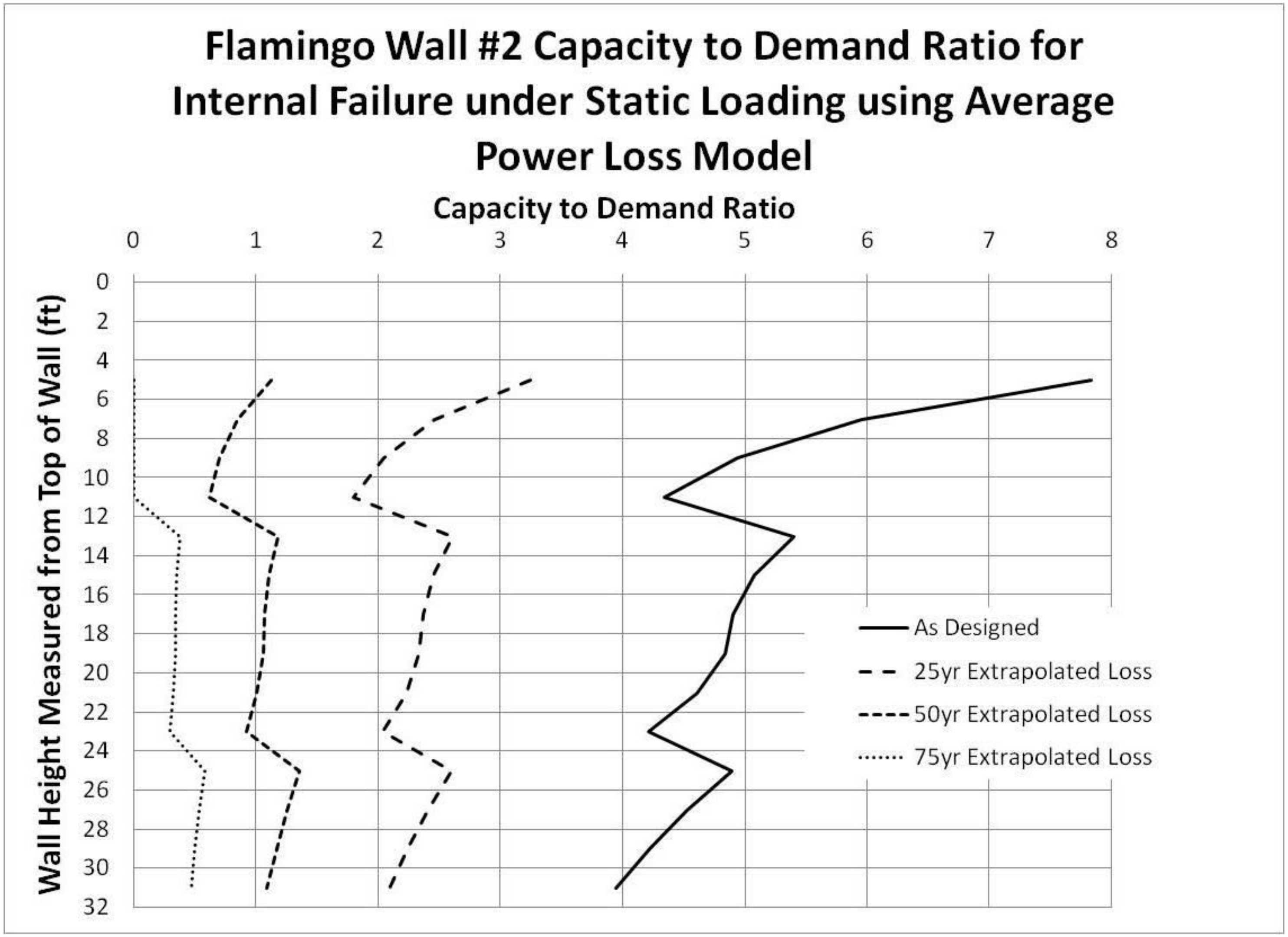

Figure 32. Flamingo Wall #2 C/D Ratio for Static Loading – Average Power Loss Model

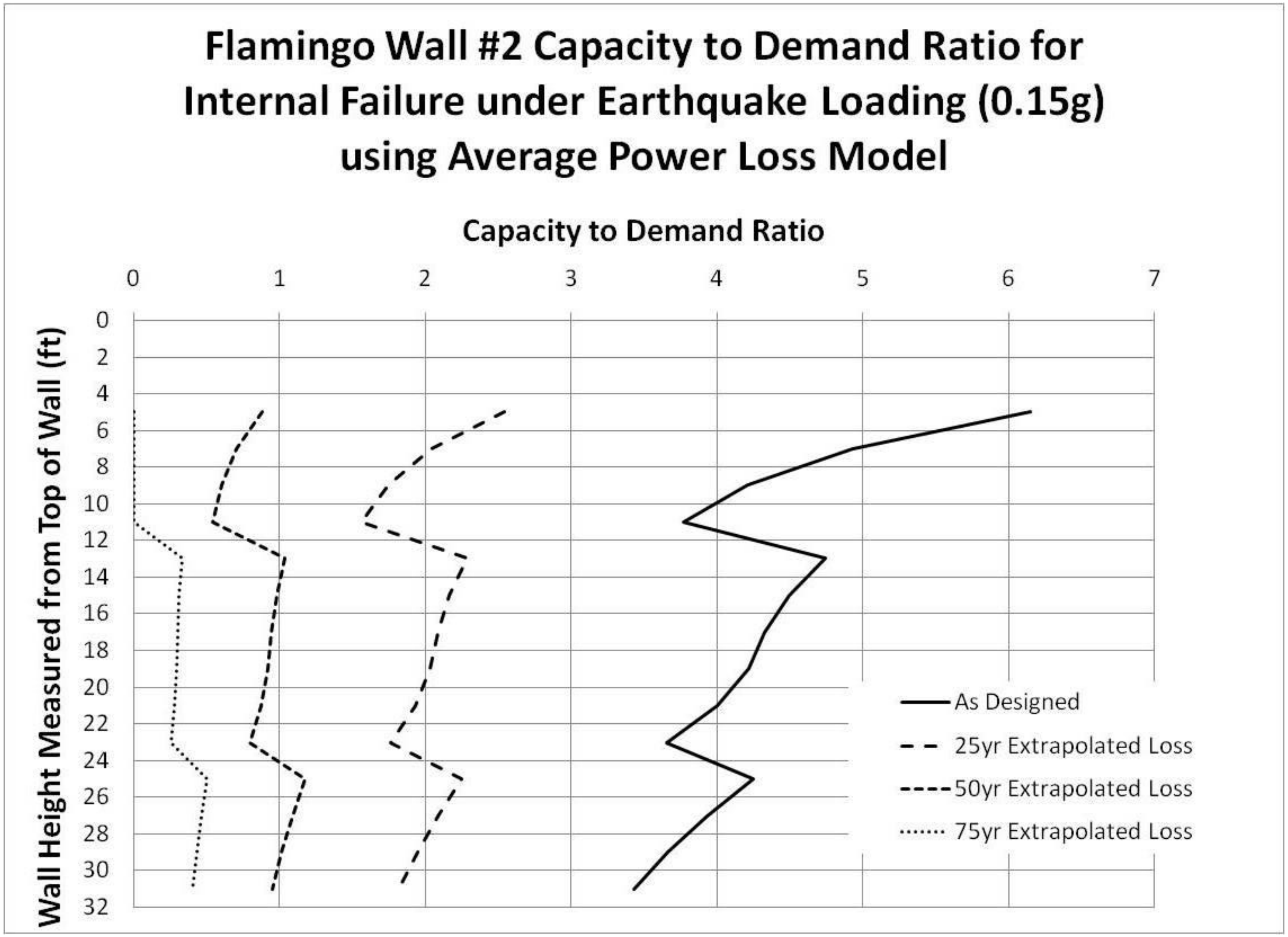

Figure 33. Flamingo Wall #2 C/D Ratio for Seismic Loading (a_{max} = 0.15g) – Average Power Loss Model

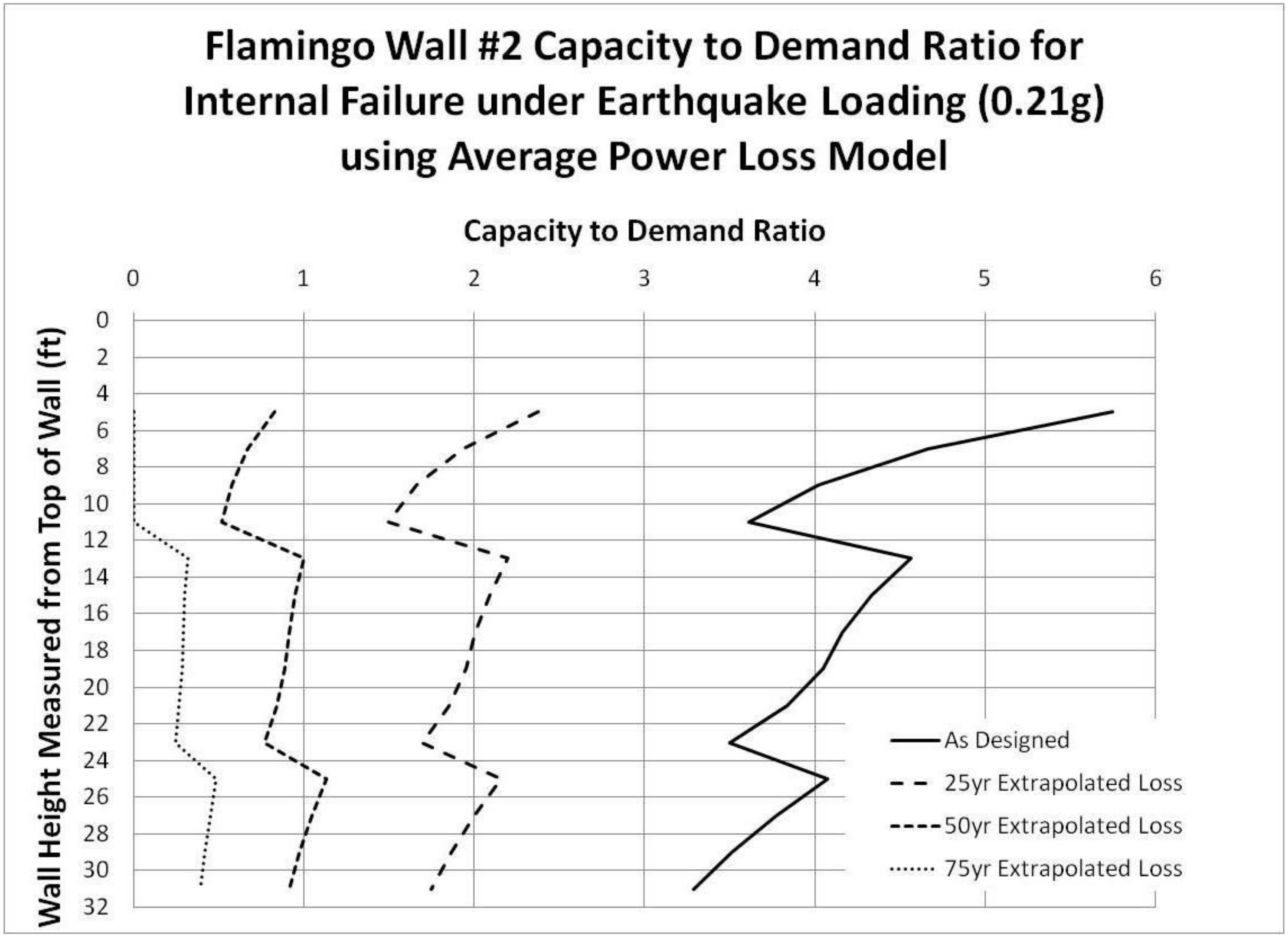

Figure 34. Flamingo Wall #2 C/D Ratio for Seismic Loading (a_{max} = 0.21g) – Average Power Loss Model

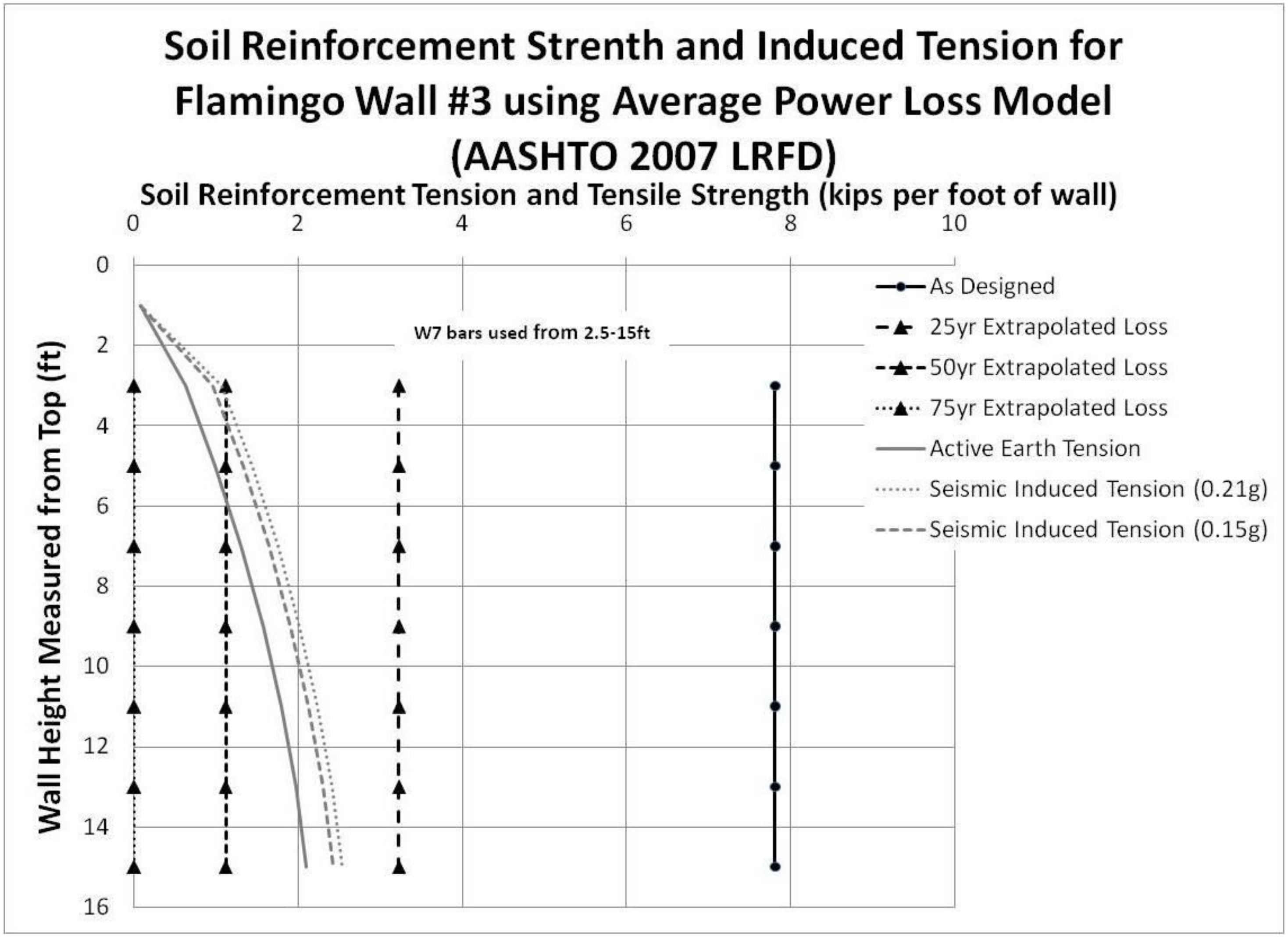

Figure 35. Soil Reinforcement Strength and Induced Tension for Flamingo Wall #3 – Average Power Loss Model

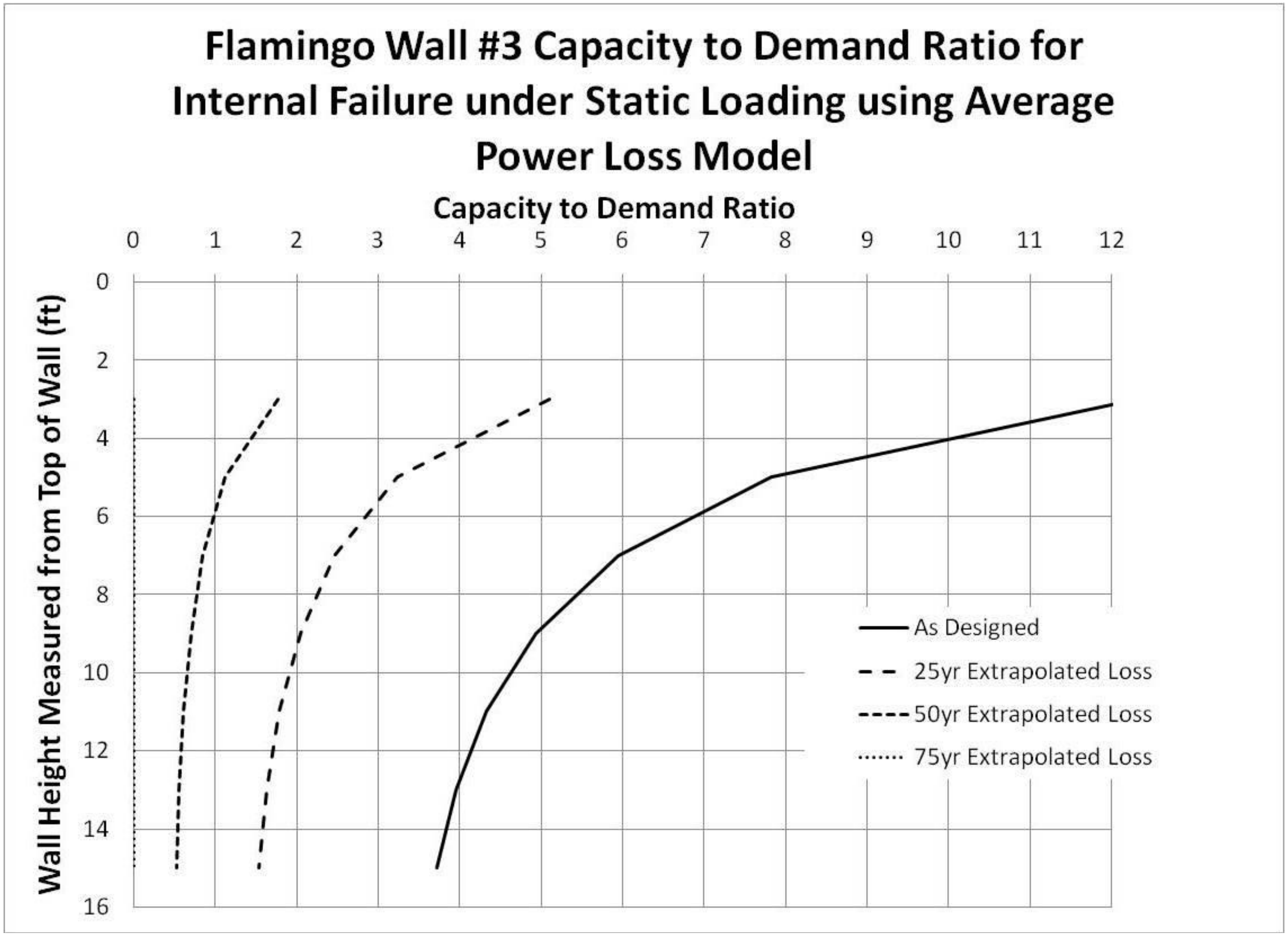

Figure 36. Flamingo Wall #3 C/D Ratio for Static Loading – Average Power Loss Model

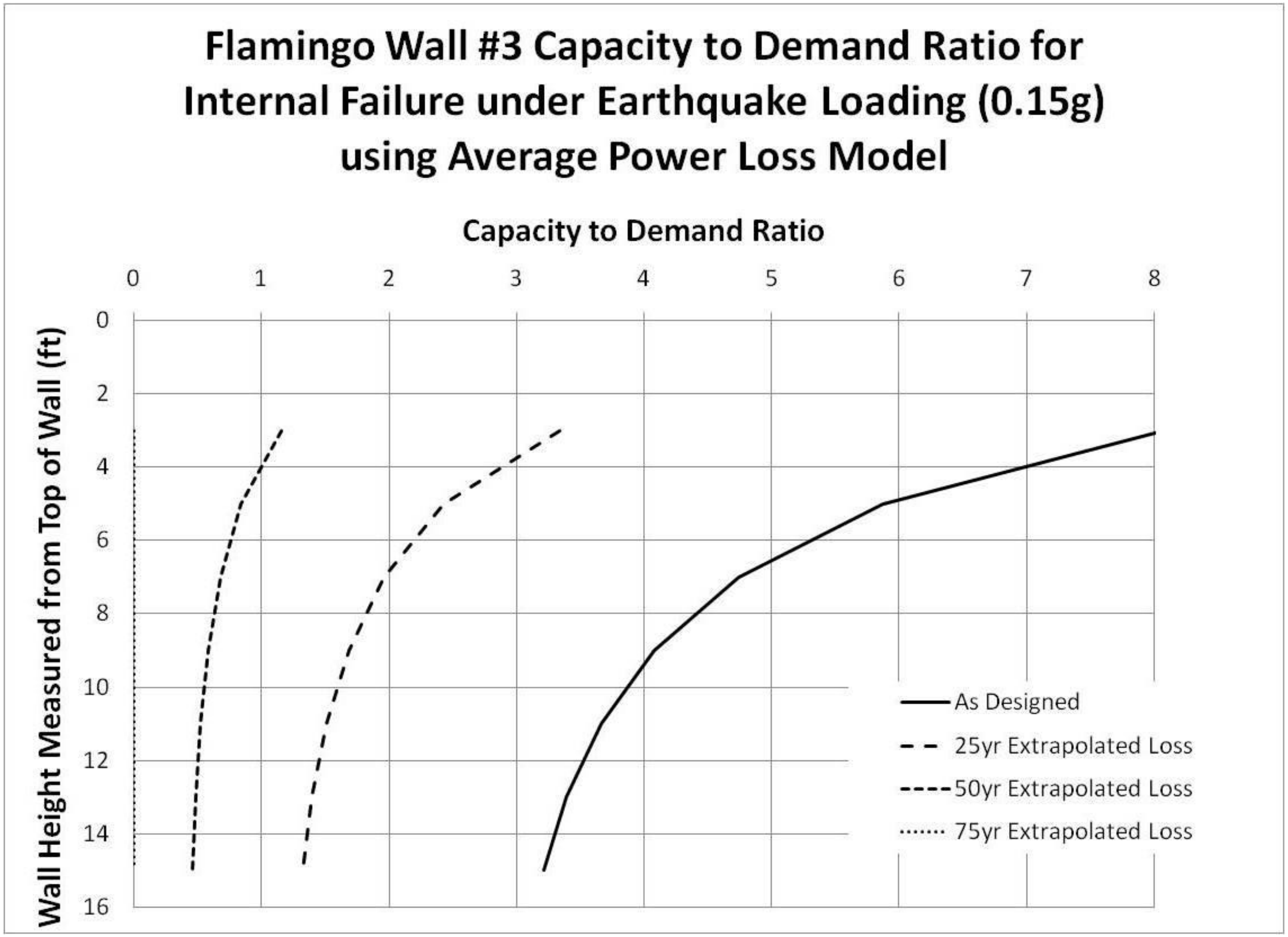

Figure 37. Flamingo Wall #3 C/D Ratio for Seismic Loading (a_{max} = 0.15g) – Average Power Loss Model

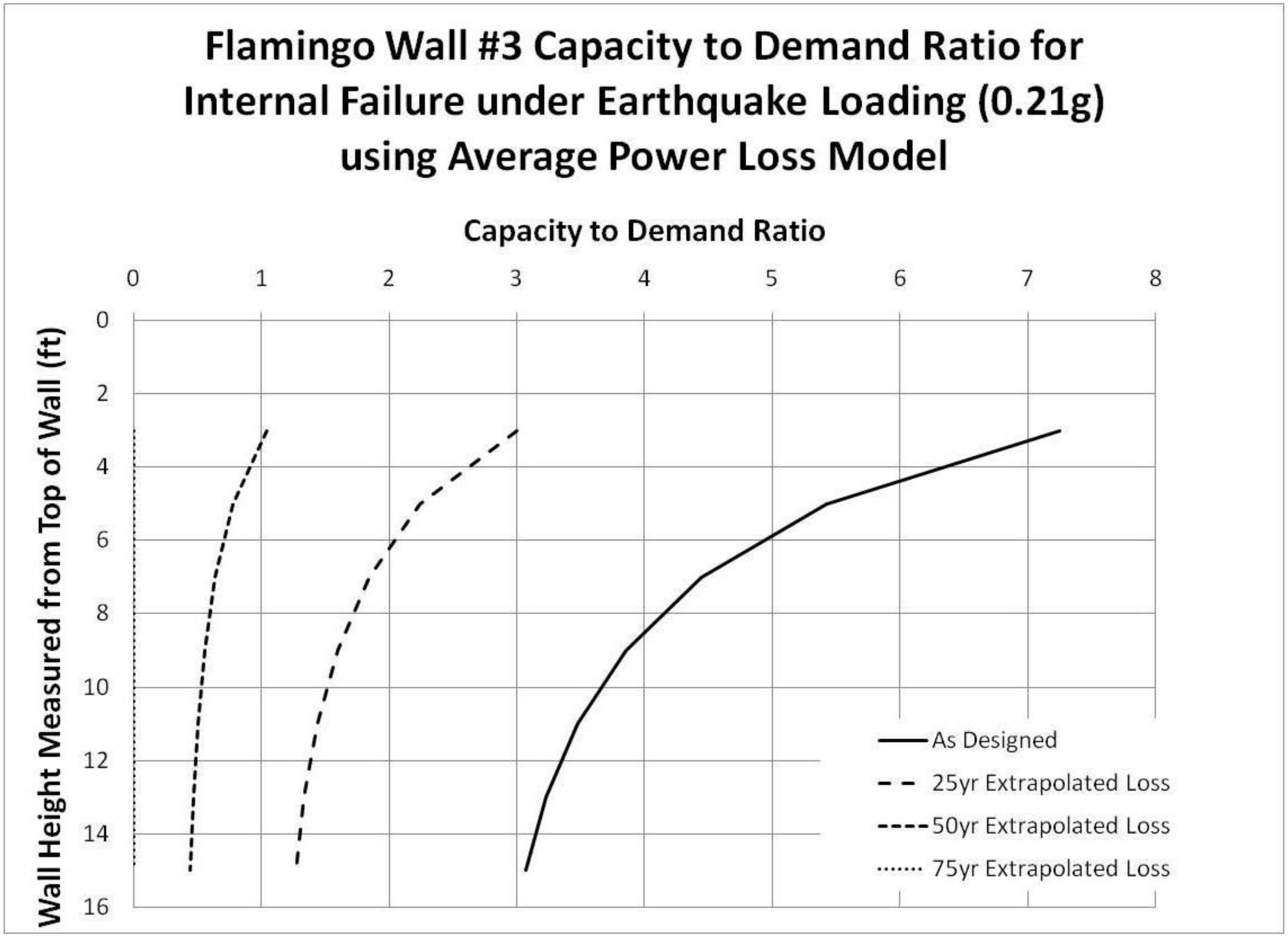

Figure 38. Flamingo Wall #3 C/D Ratio for Seismic Loading (a_{max} = 0.21g) – Average Power Loss Model

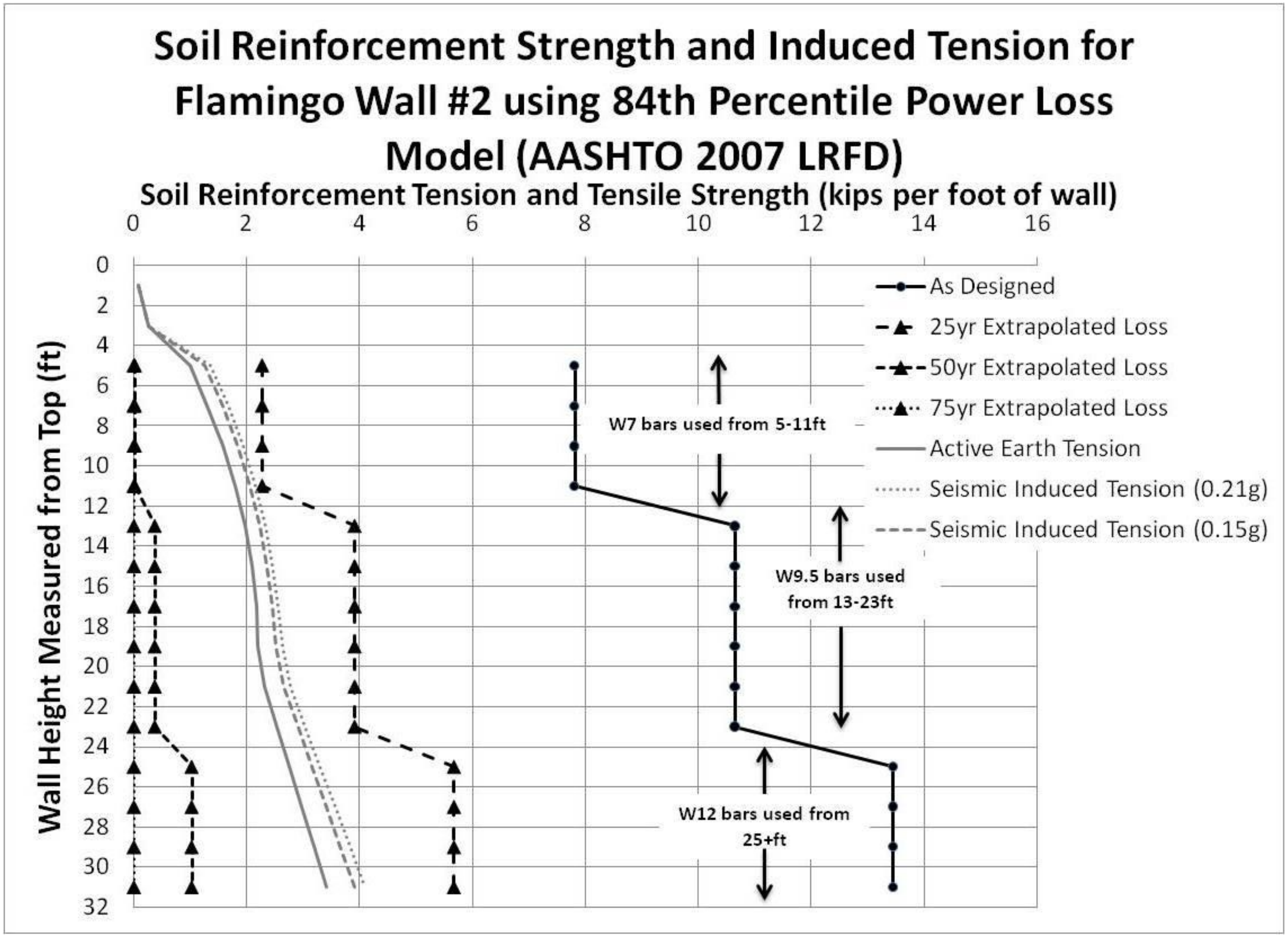

Figure 39. Soil Reinforcement Strength and Induced Tension for Flamingo Wall #2 – 84th Percentile Power Loss Model

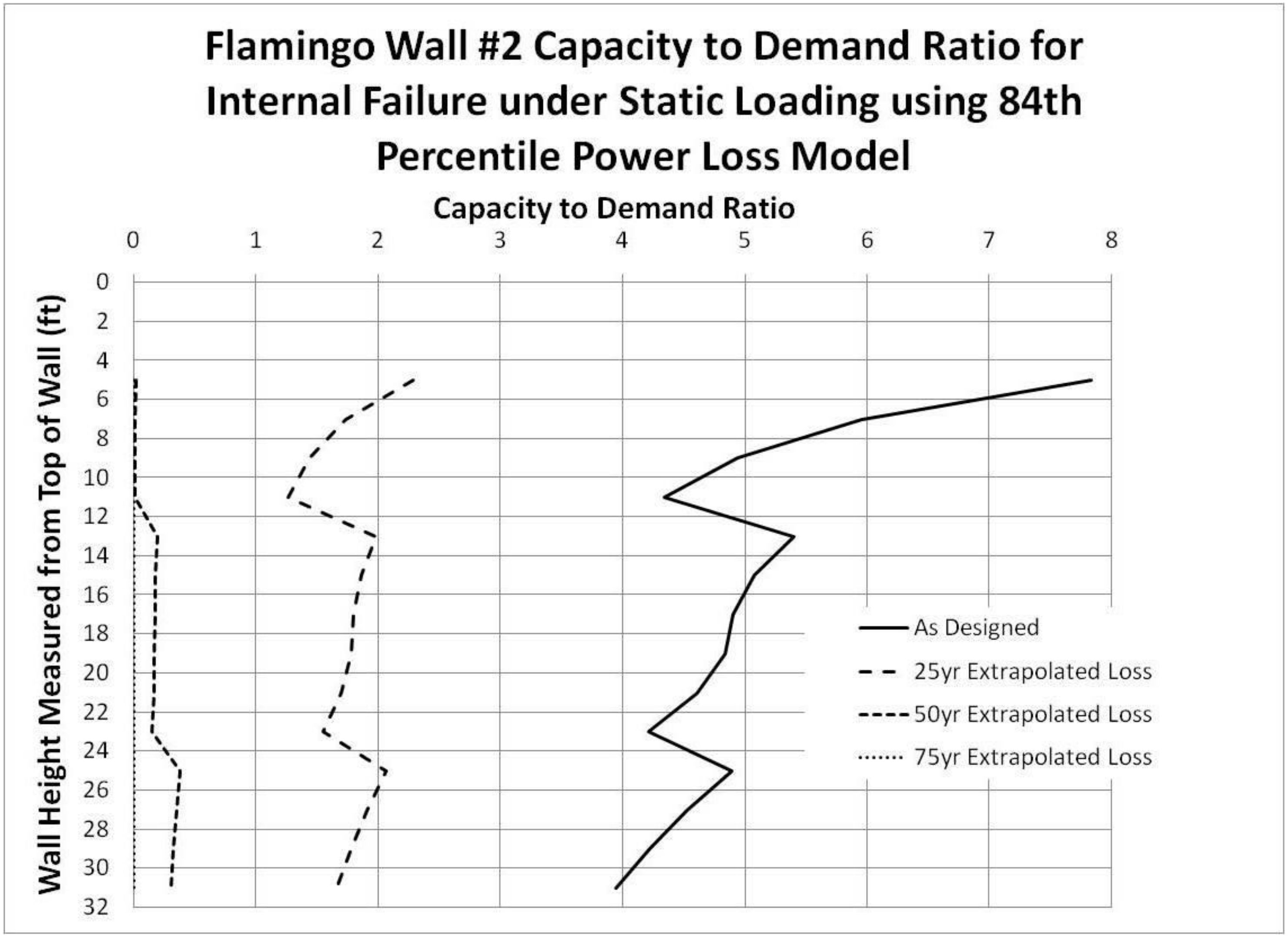

Figure 40. Flamingo Wall #2 C/D Ratio for Static Loading – 84th Percentile Power Loss Model

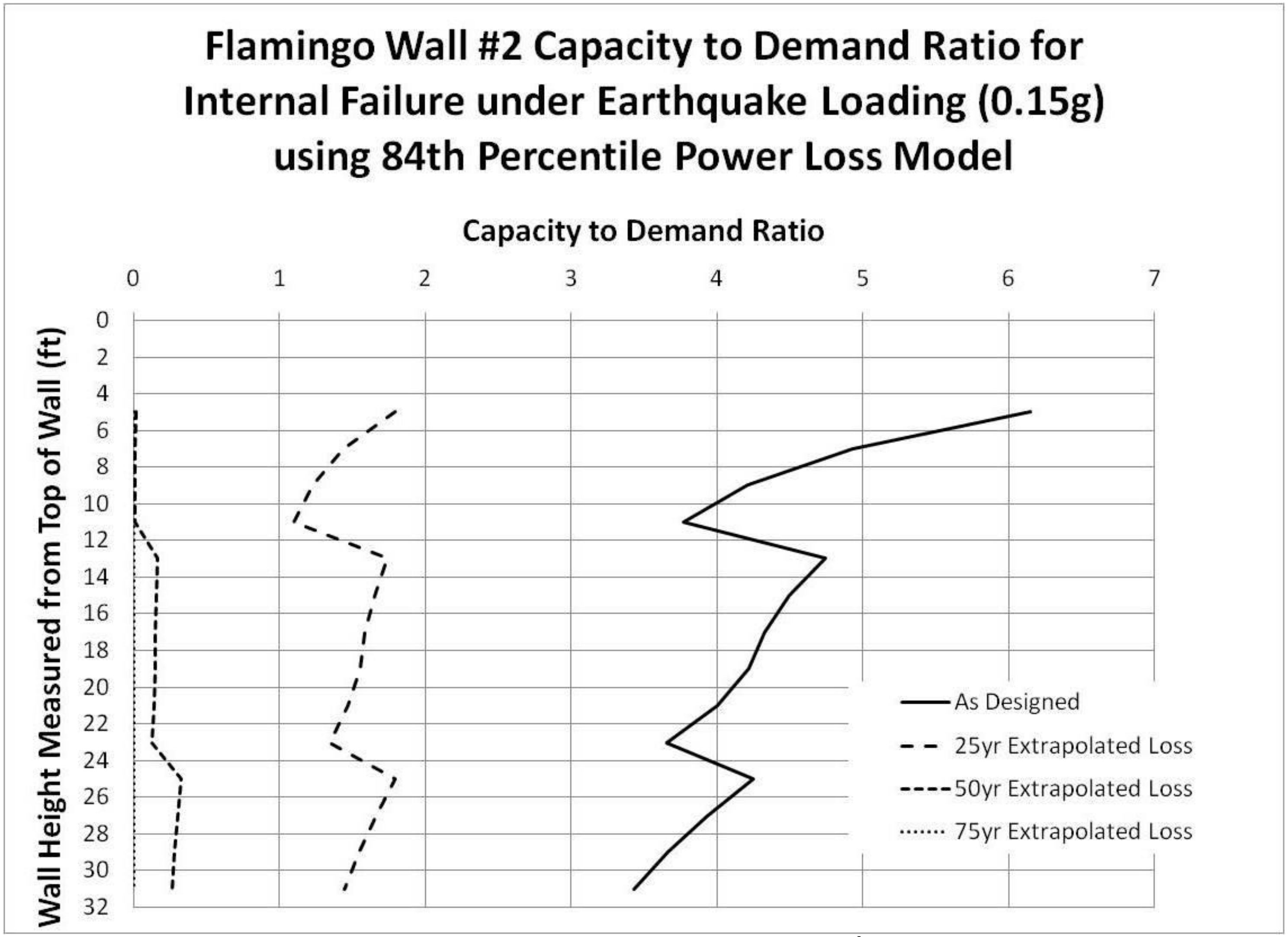

Figure 41. Flamingo Wall #2 C/D Ratio for Seismic Loading (a_{max} = 0.15g) – 84th Percentile Power Loss Model

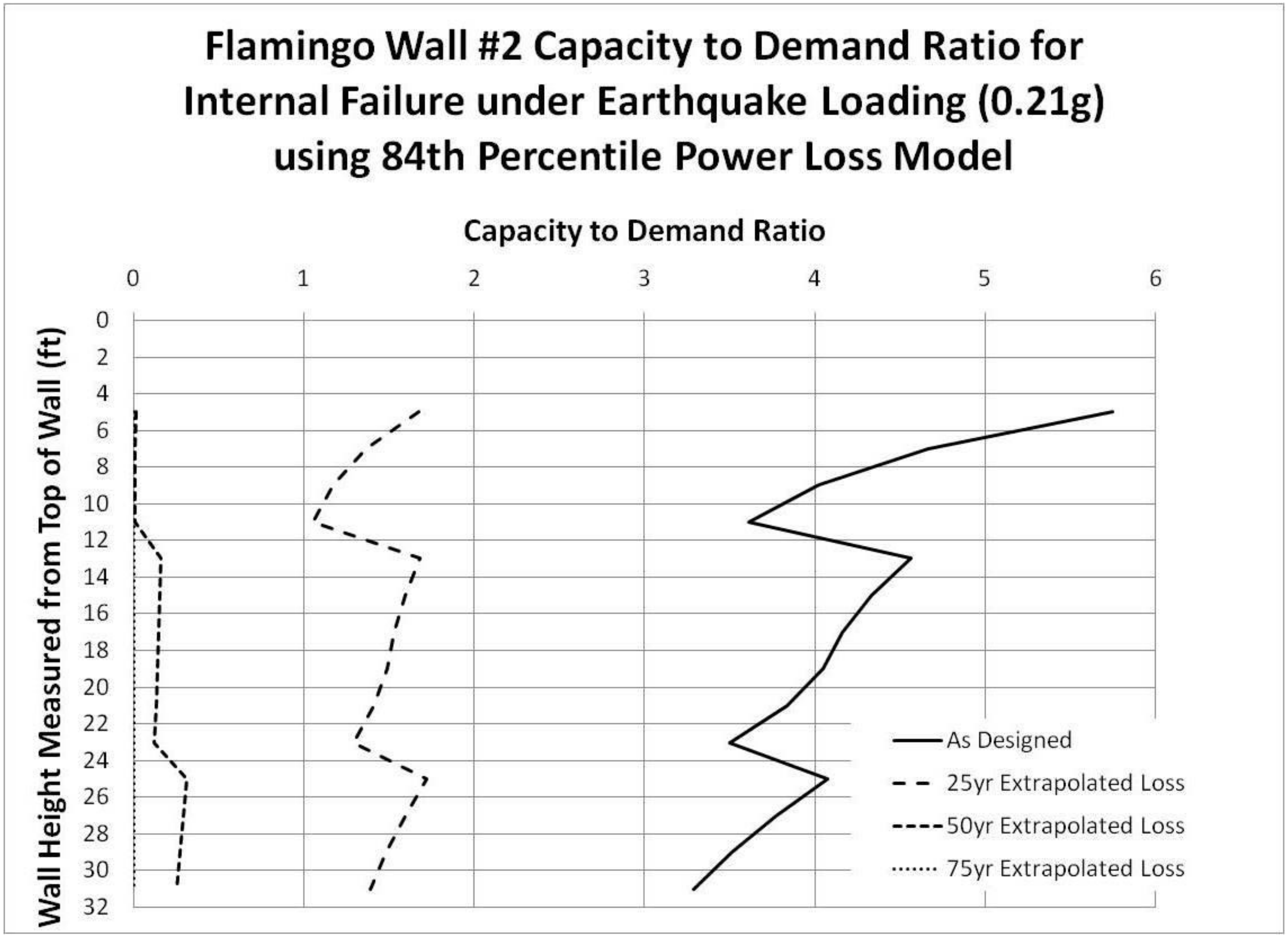

Figure 42. Flamingo Wall #2 C/D Ratio for Seismic Loading (a_{max} = 0.21g) – 84th Percentile Power Loss Model

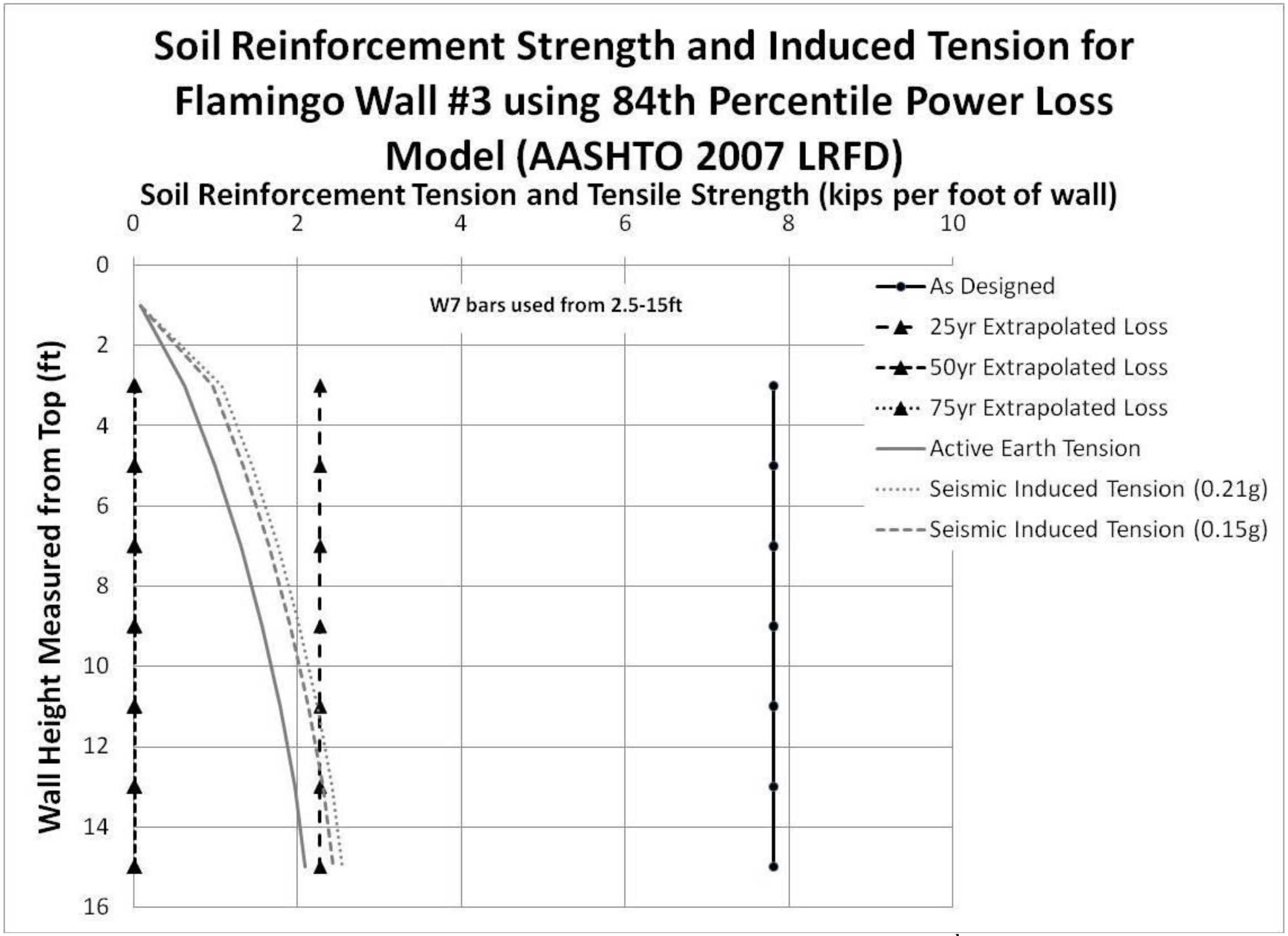

Figure 43. Soil Reinforcement Strength and Induced Tension for Flamingo Wall #3 – 84th Percentile Power Loss Model

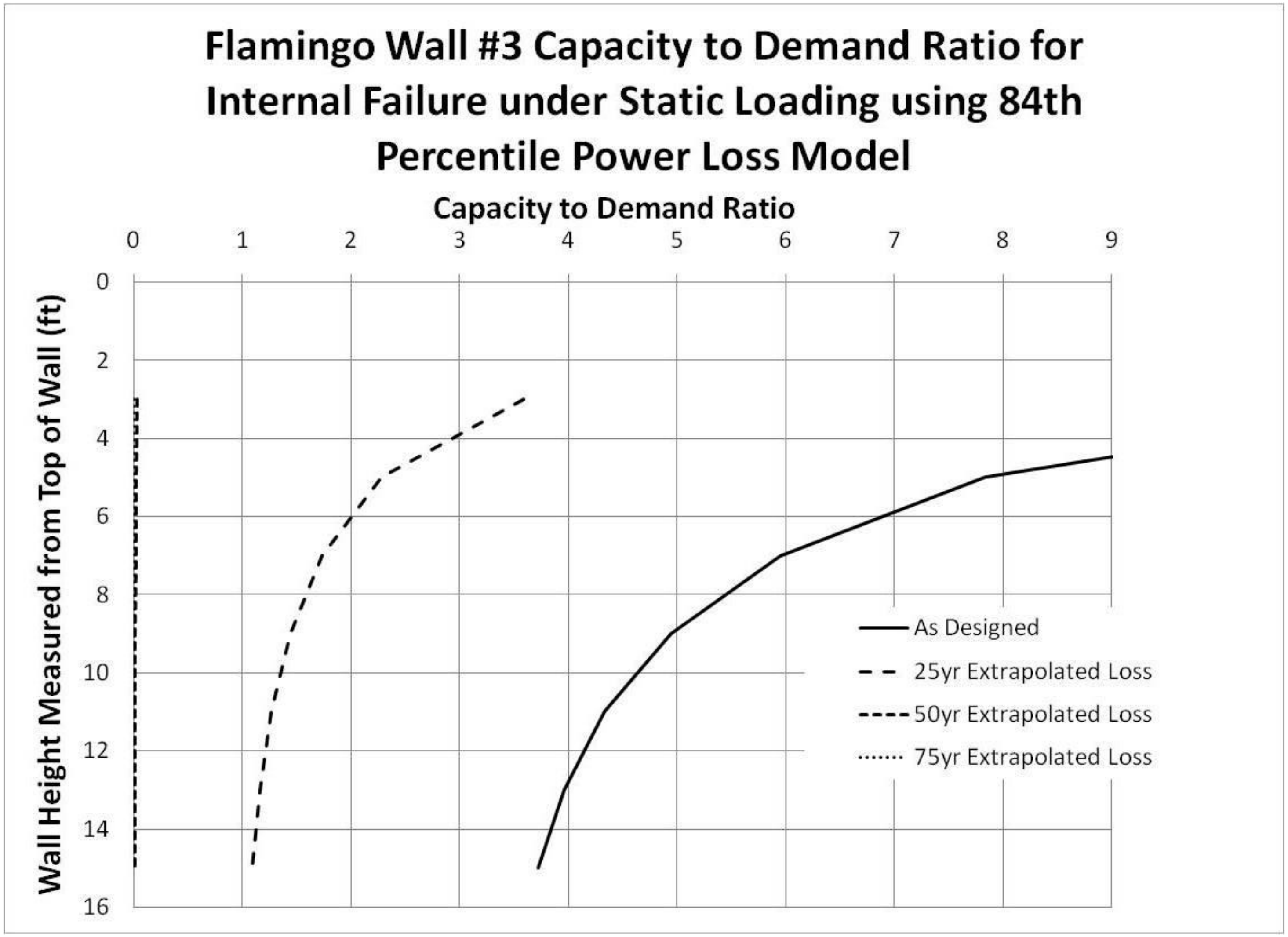

Figure 44. Flamingo Wall #3 C/D Ratio for Static Loading – 84th Percentile Power Loss Model

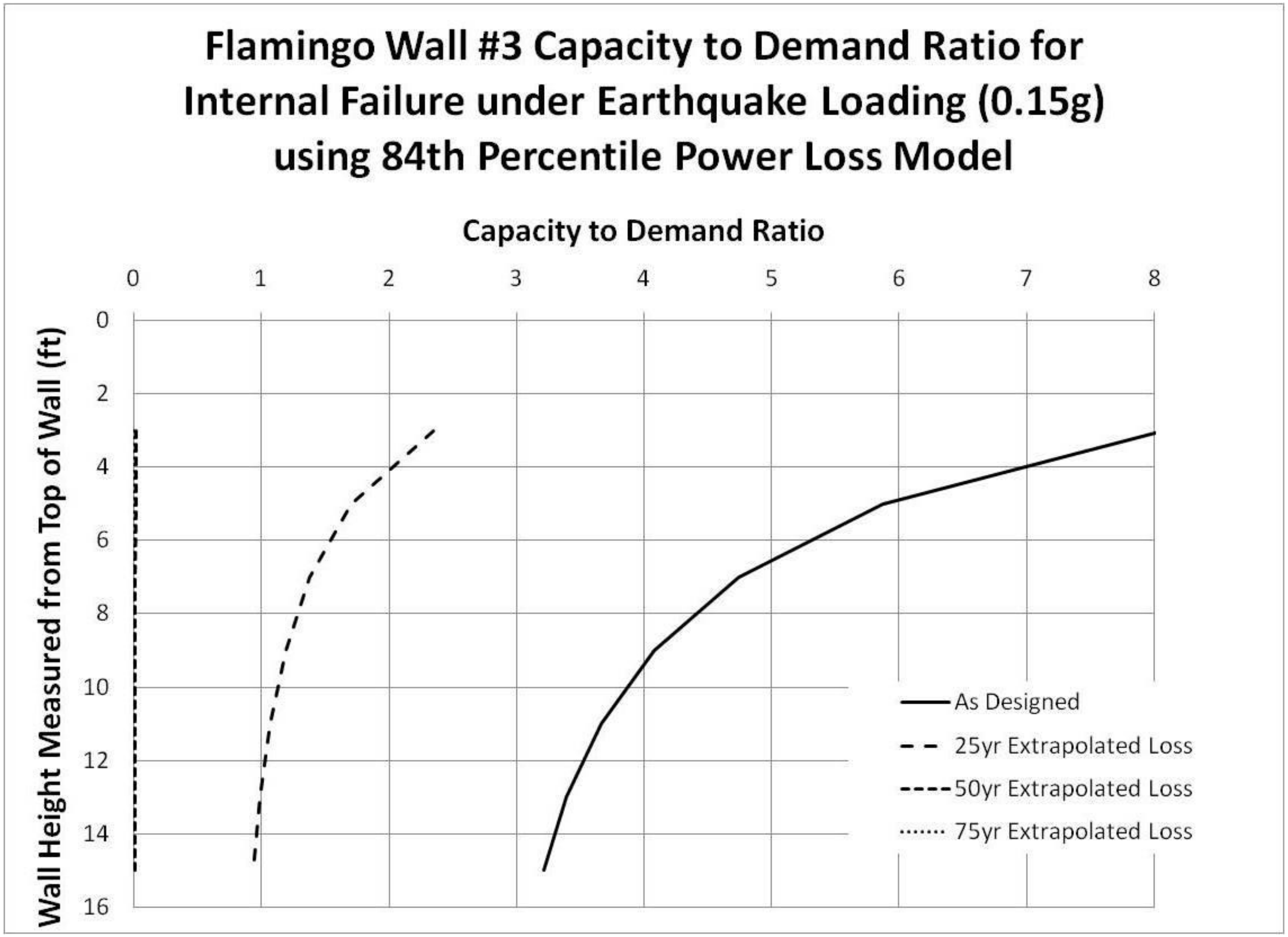

Figure 45. Flamingo Wall #3 C/D Ratio for Seismic Loading (a_{max} = 0.15g) – 84th Percentile Power Loss Model

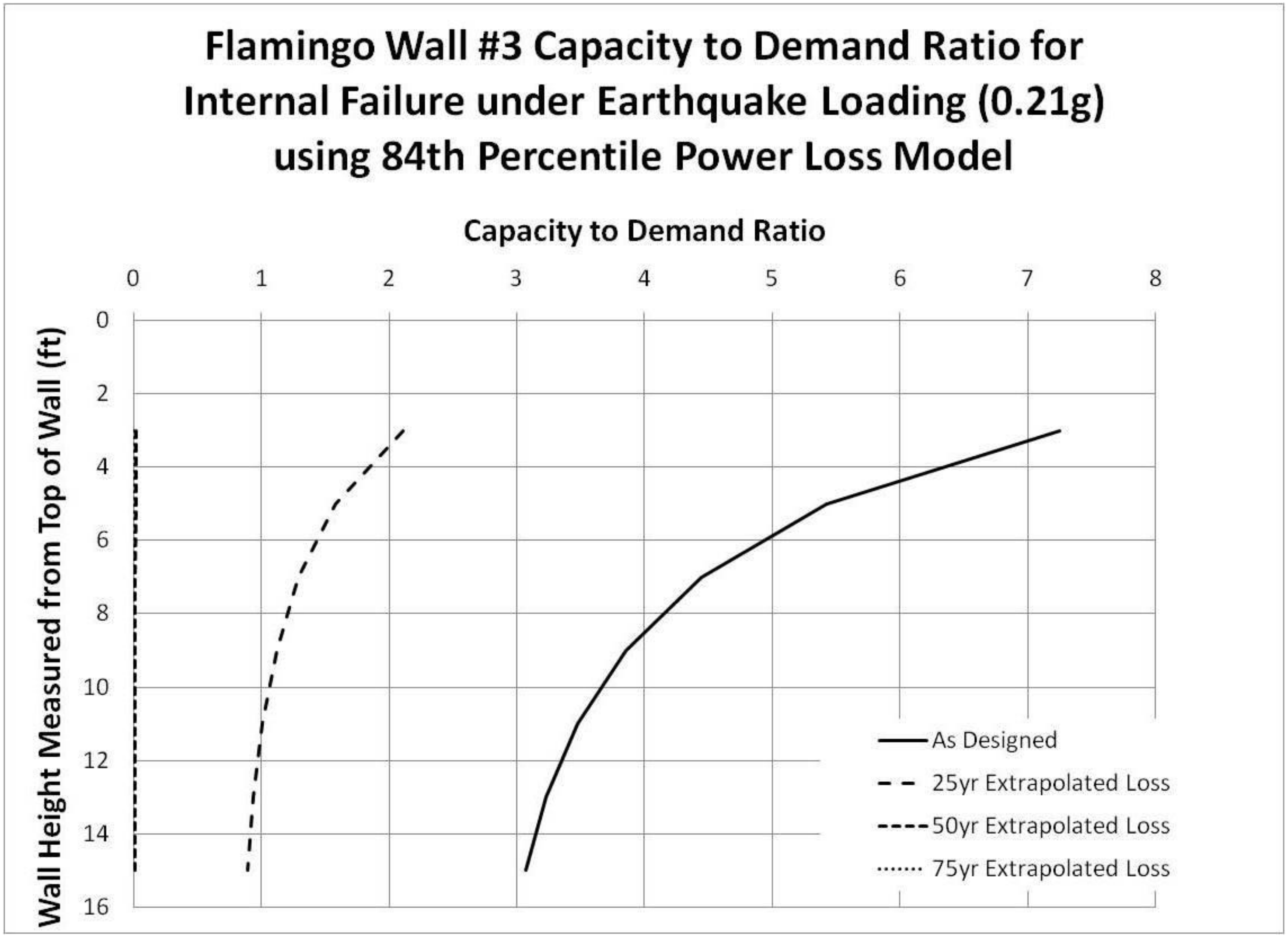

Figure 46. Flamingo Wall #3 C/D Ratio for Seismic Loading (a_{max} = 0.21g) – 84th Percentile Power Loss Model

Figure 47. Cheyenne MSE Wall Corrosion Investigation - Highly Corroded Reinforcing Strip

Figure 48. Cheyenne MSE Wall Corrosion Investigation - Reinforcing Strips in Pile of Debris

Figure 49. Cheyenne MSE Wall Corrosion Investigation - Styrofoam at Top End of Panel

Figure 50. Cheyenne MSE Wall Corrosion Investigation - Corroded Steel Facing Connection

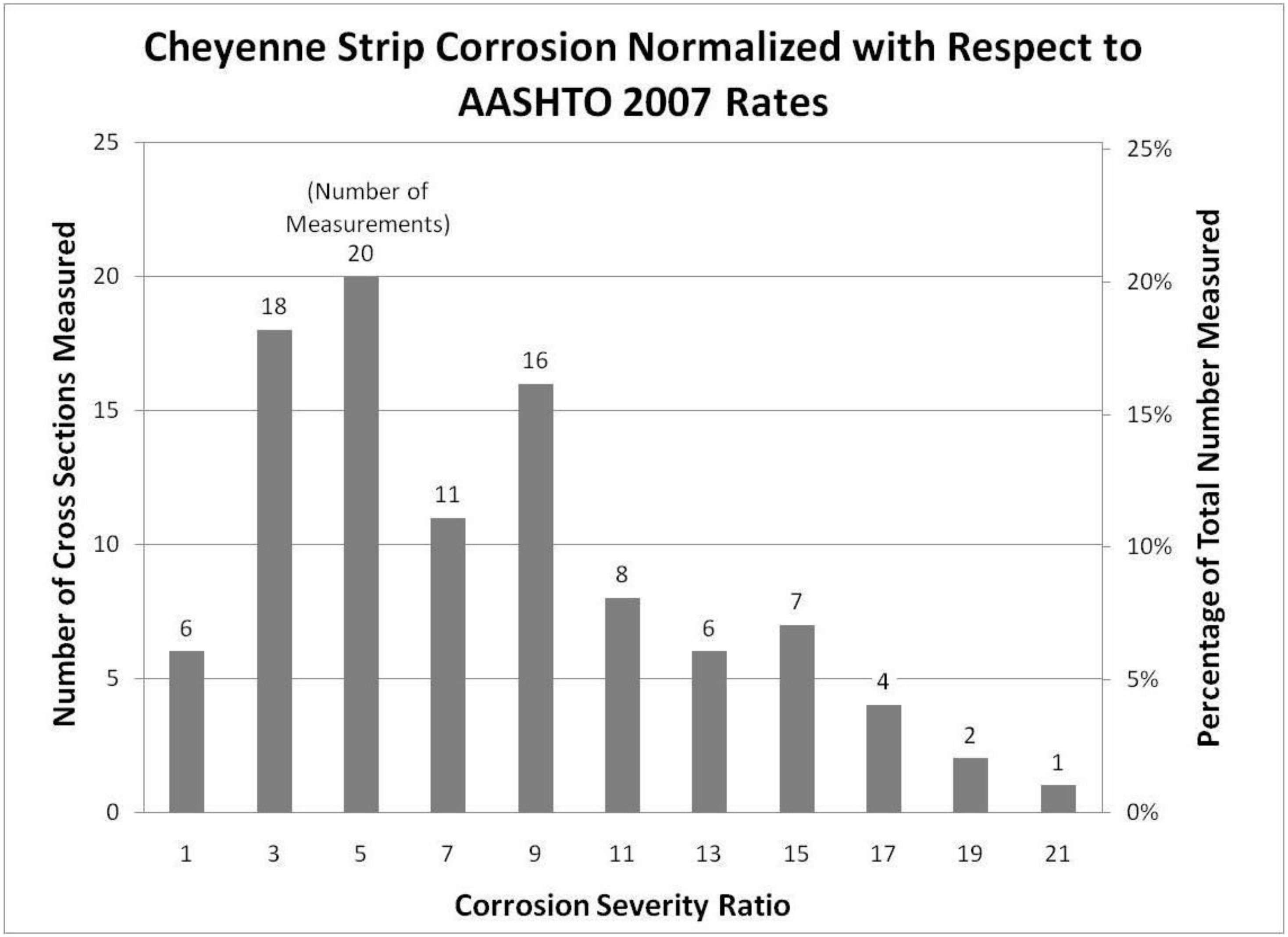

Figure 51. Distribution of Corrosion Rates with Respect to AASHTO Design Rates (2007) for the Cheyenne MSE Wall

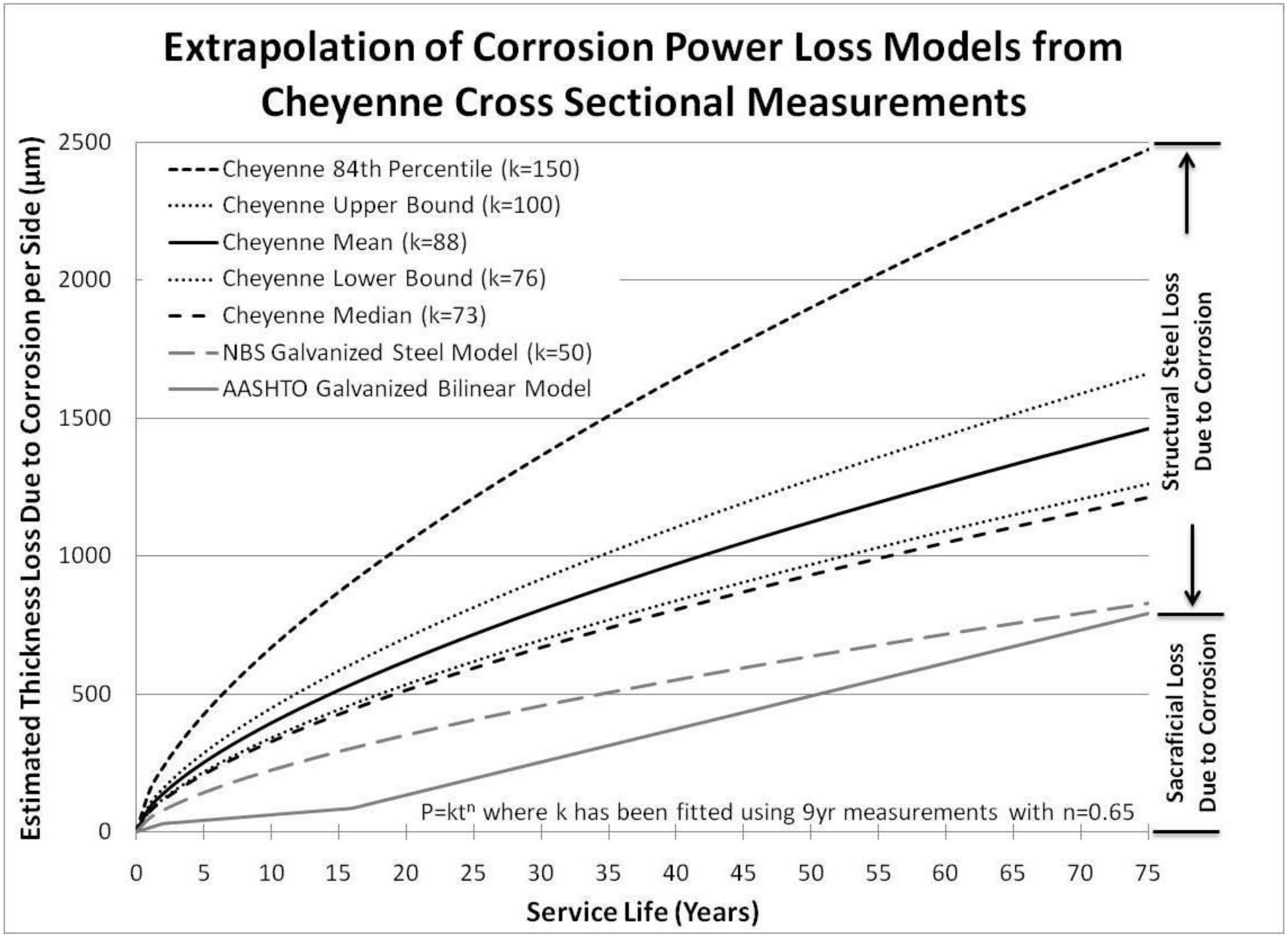

Figure 52. Extrapolation of Corrosion Loss for Cheyenne Cross Sectional Measurements

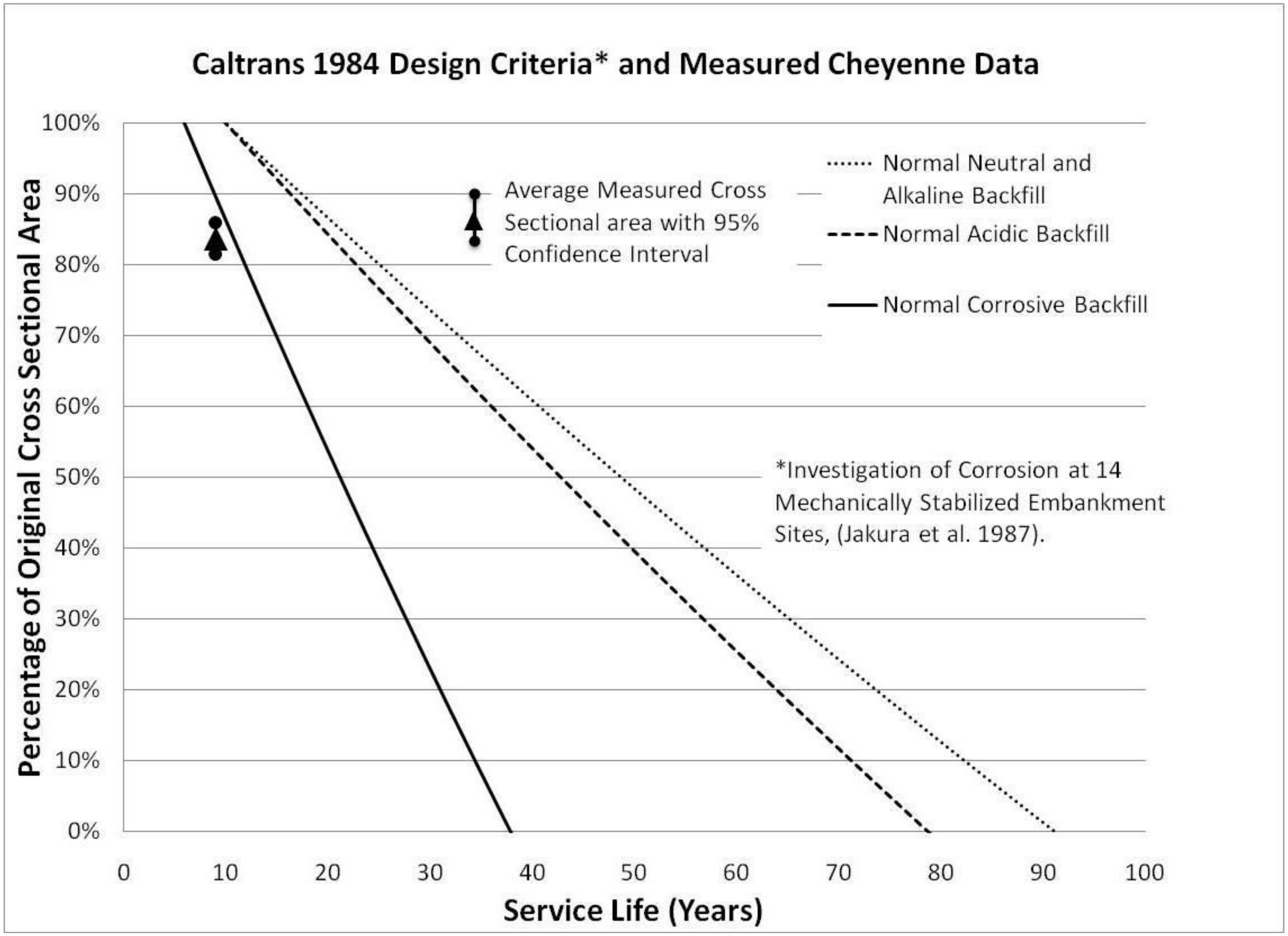

Figure 53. Potential Corrosion Loss Predicted by Cheyenne Corroded Section Measurements Compared to Caltrans (1984) Model

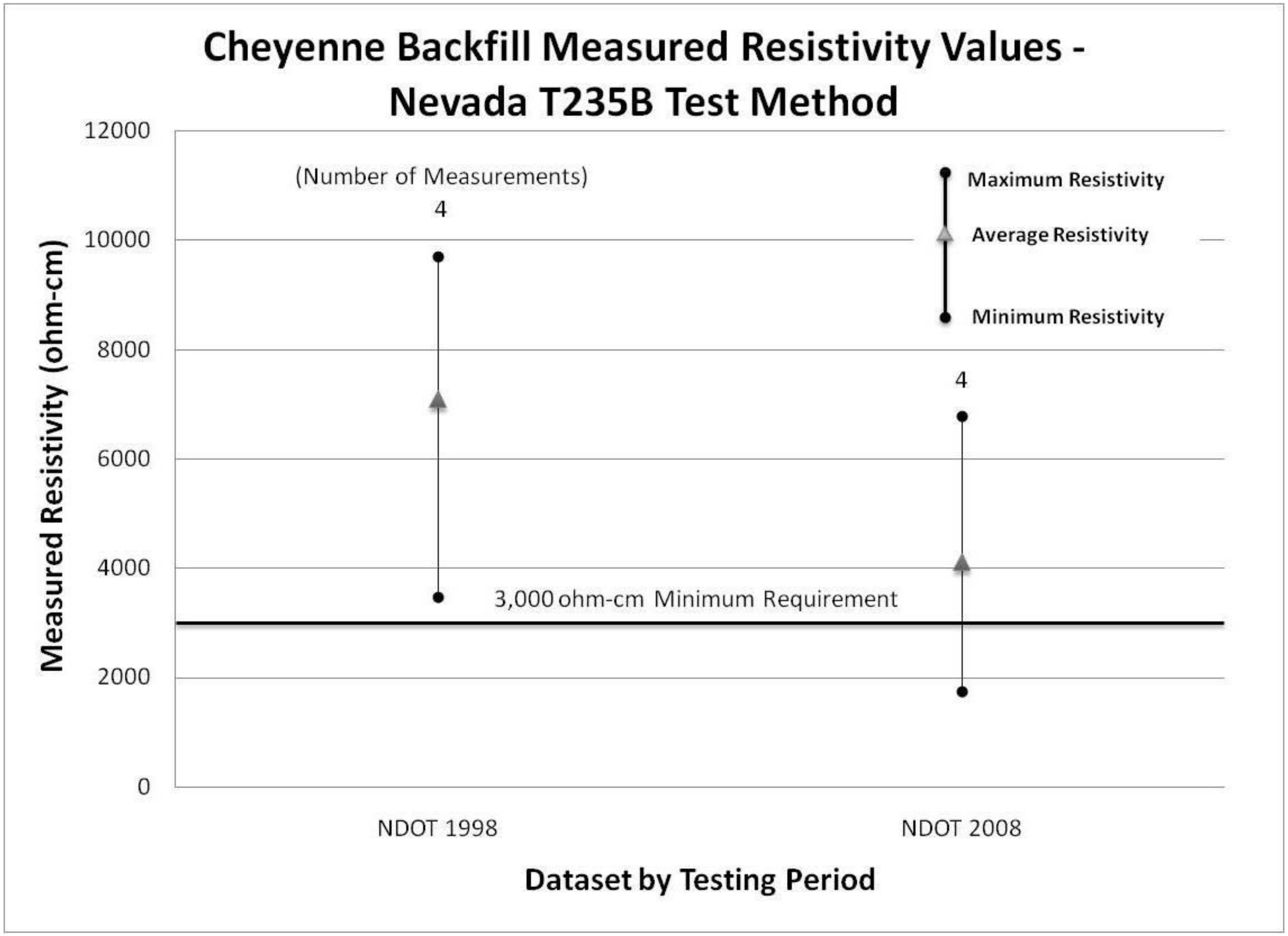

Figure 54. Cheyenne Backfill Measured Soil Resistivity Values – Nevada T235B Method

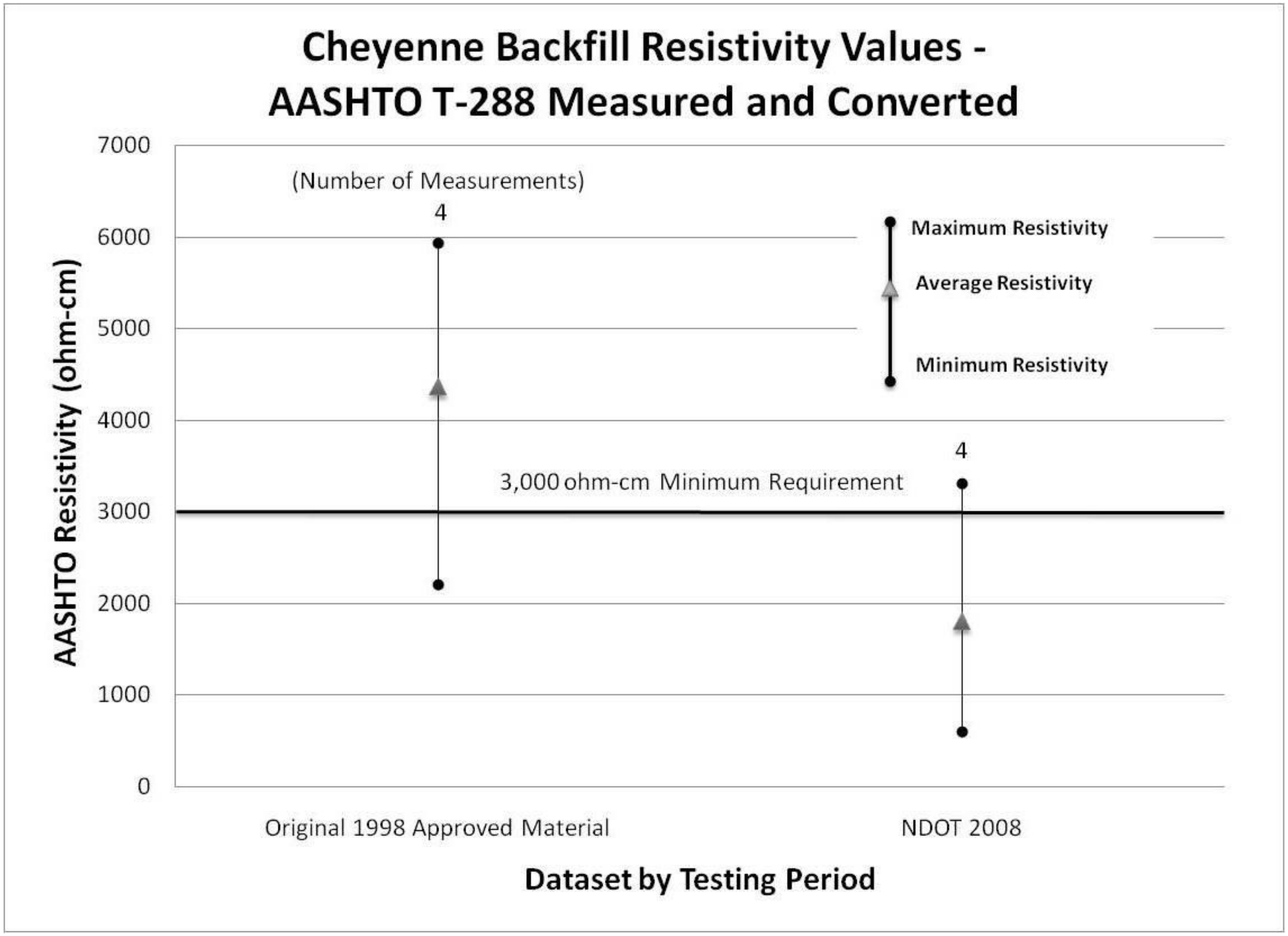

Figure 55. Cheyenne Backfill AASTHO Soil Resistivity Values – Measured and Converted

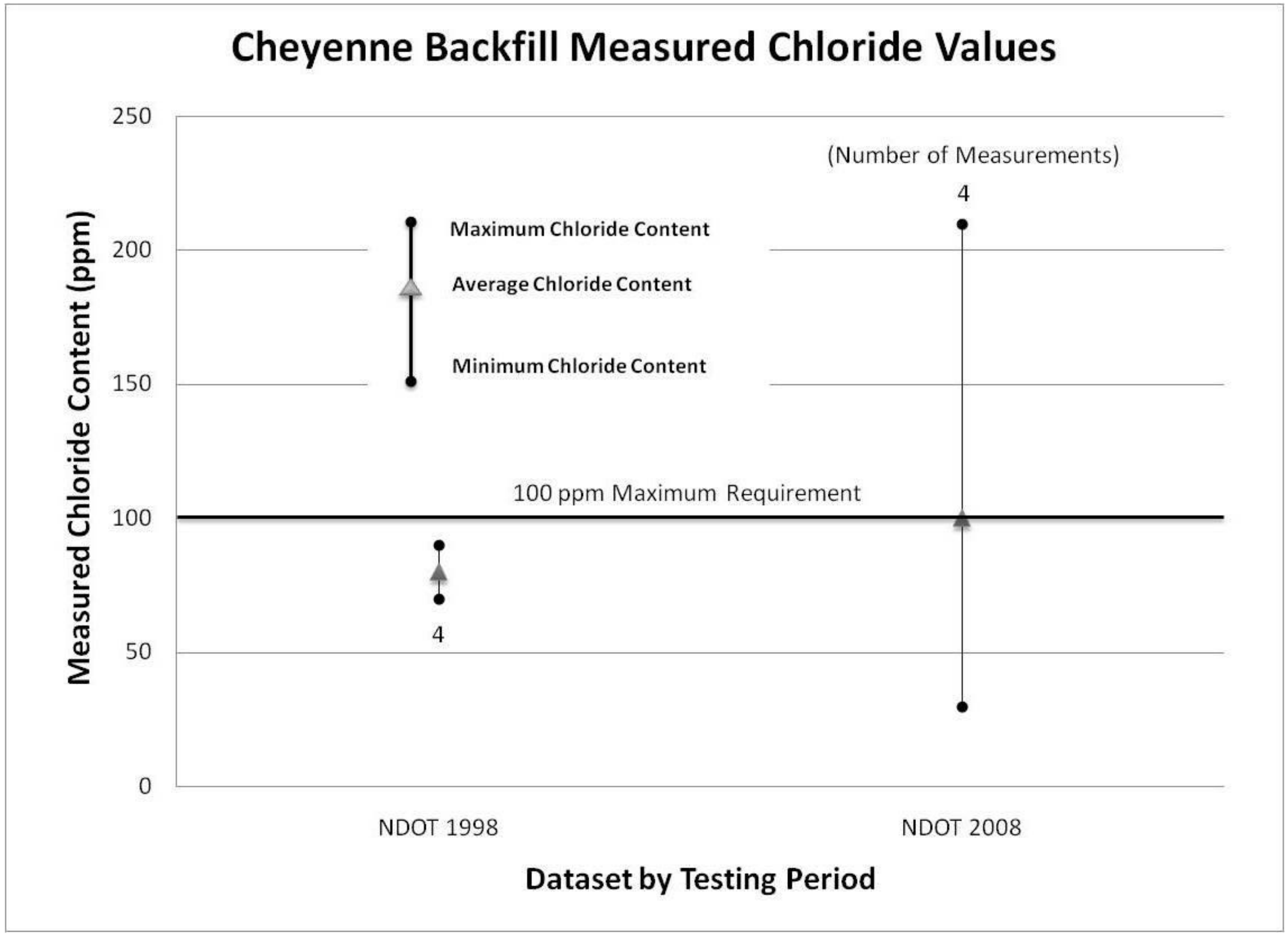

Figure 56. Cheyenne Backfill Measured Chloride Content Values

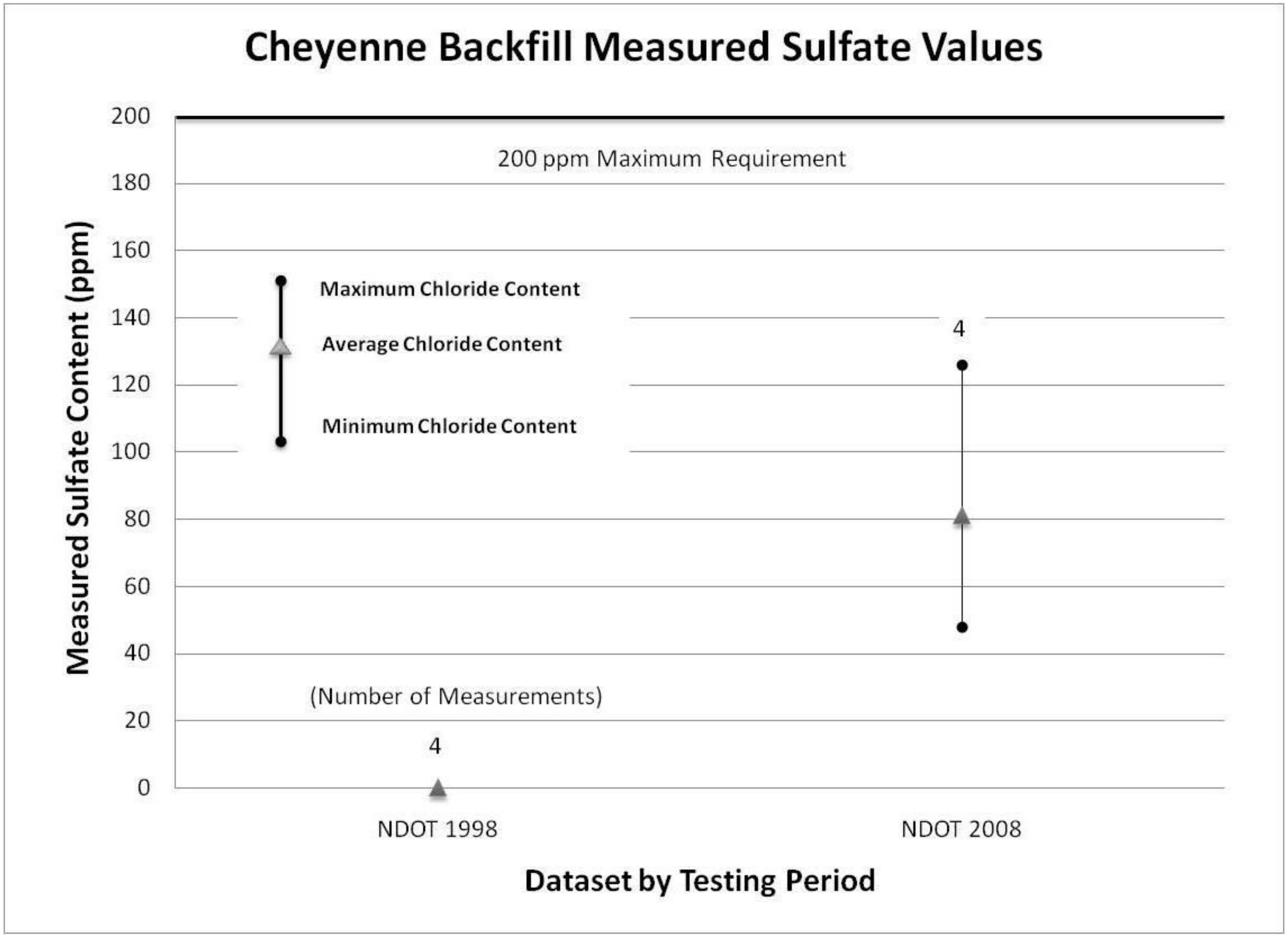

Figure 57. Cheyenne Backfill Measured Sulfate Content Values

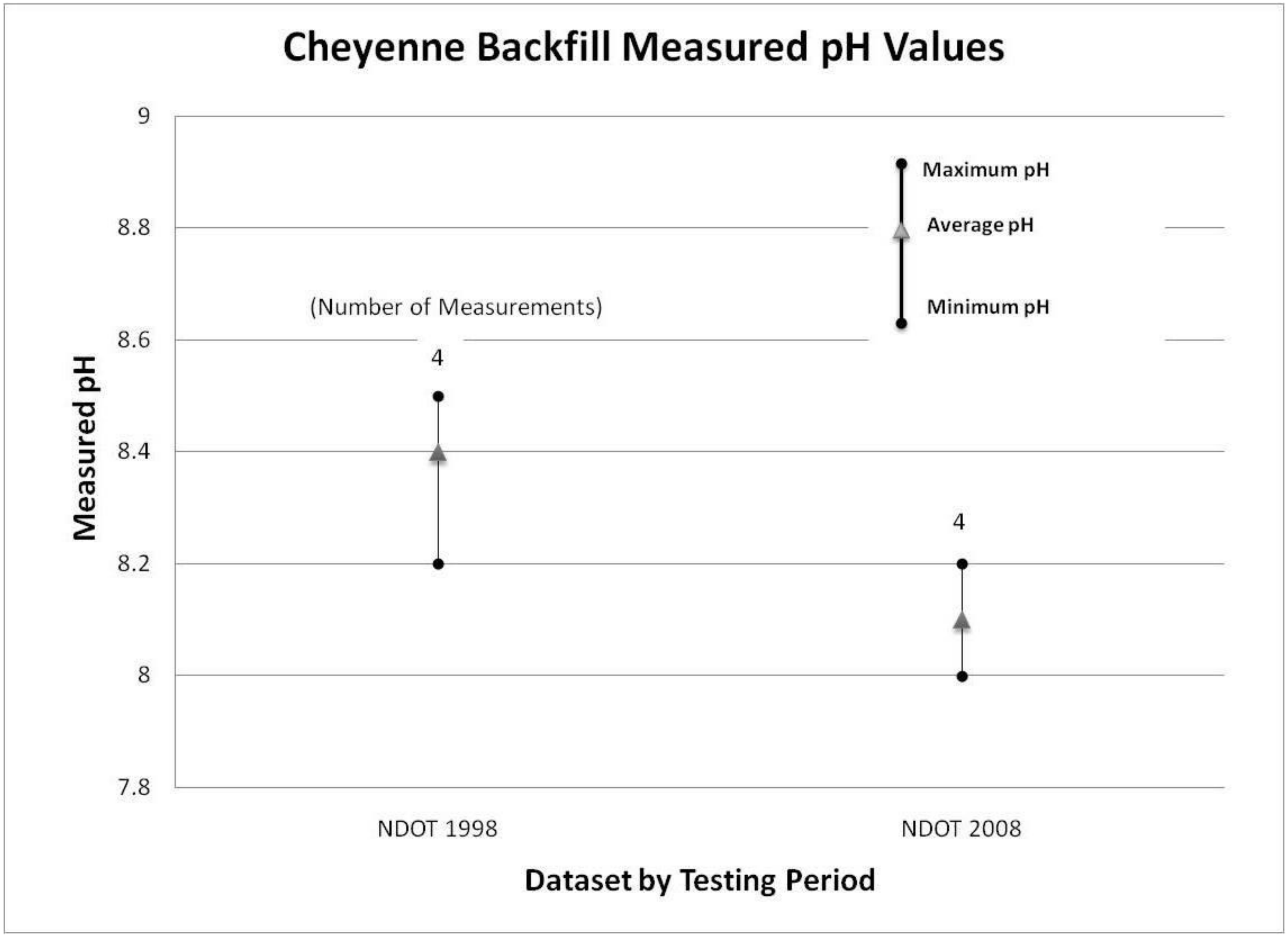

Figure 58. Cheyenne Backfill Measured pH Values

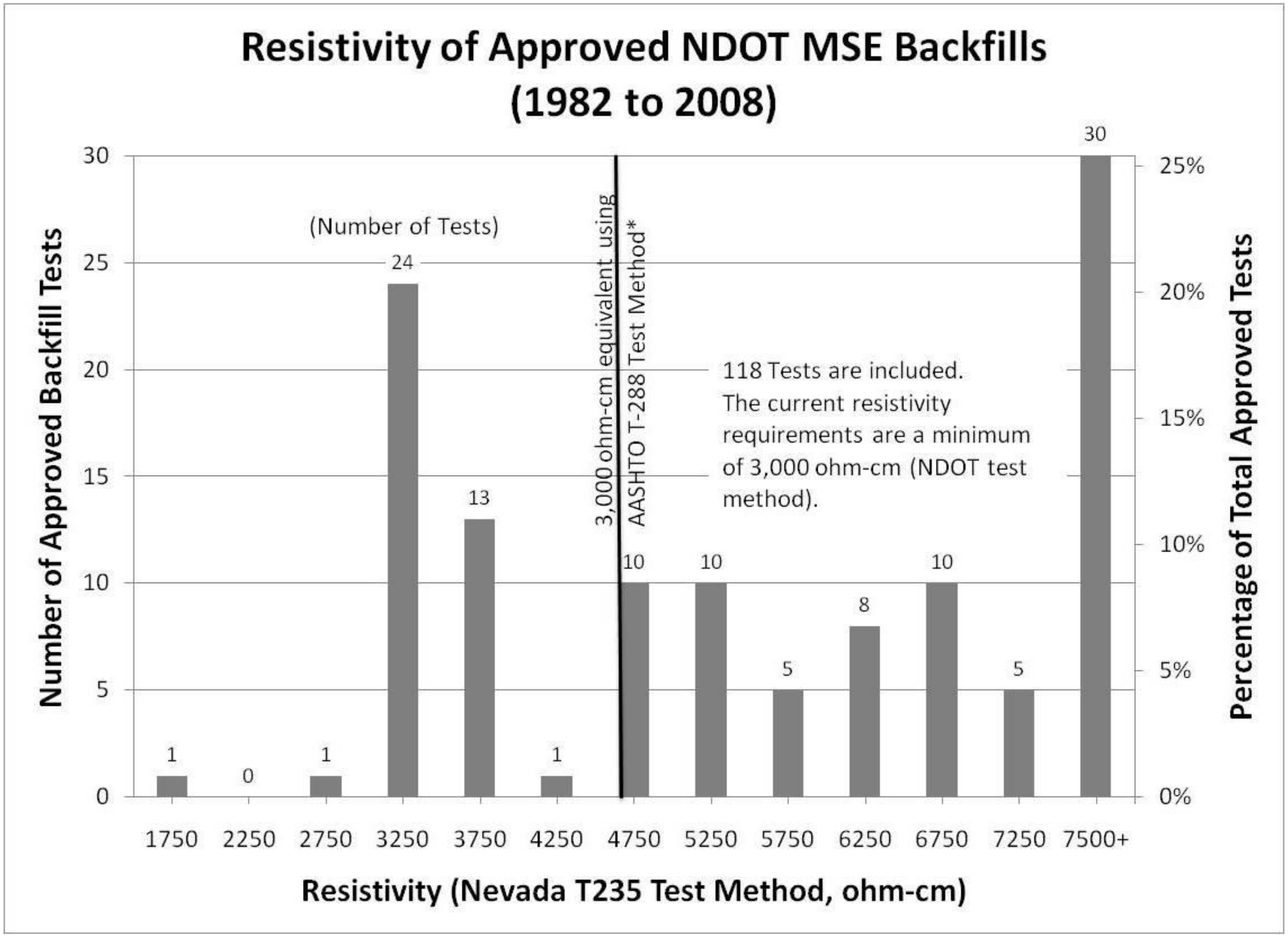

Figure 59. Resistivity of Approved Backfill in NDOT MSE Walls

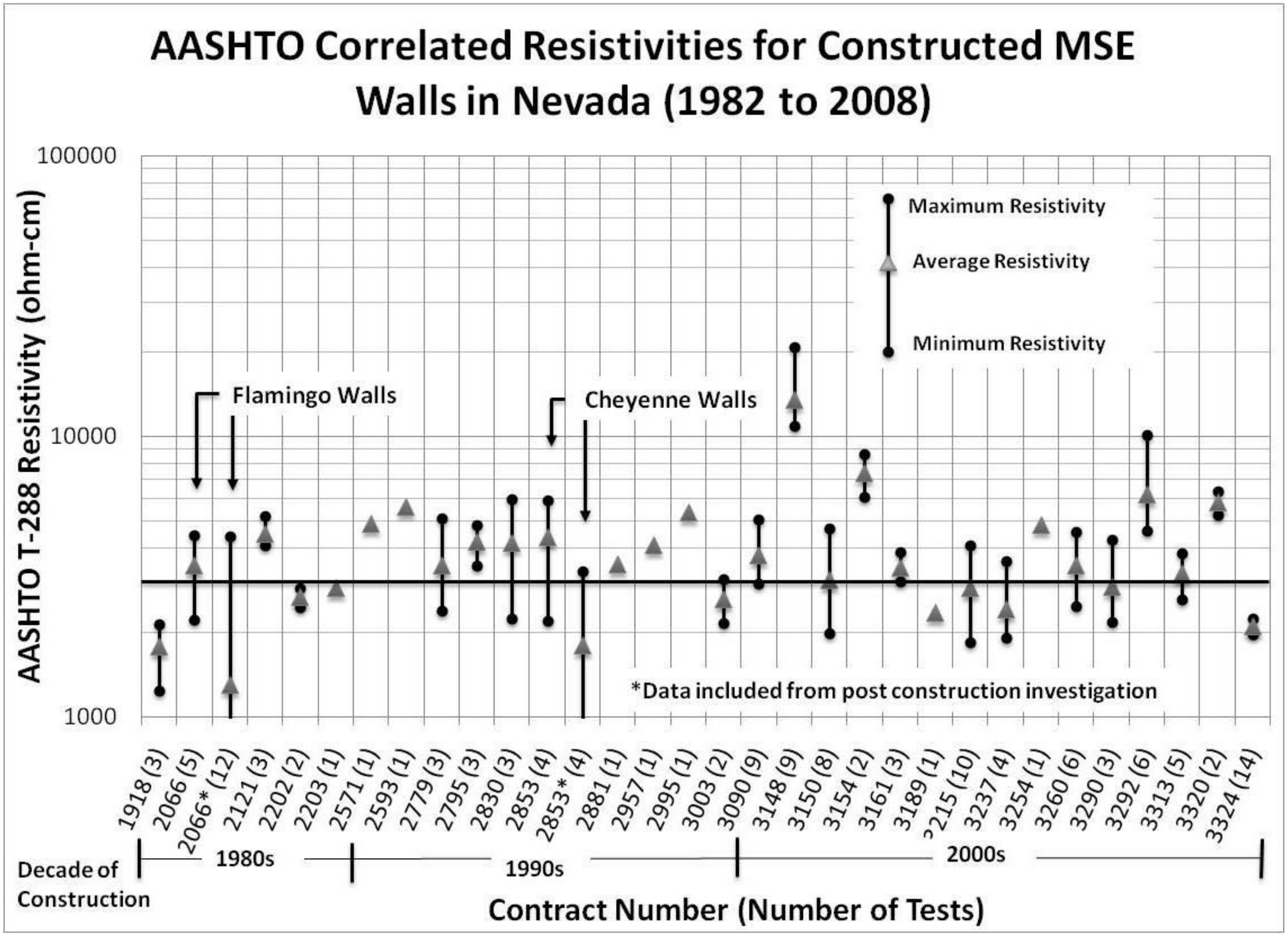

Figure 60. AASHTO Resistivity Data for MSE Wall Contracts in Nevada – All Districts

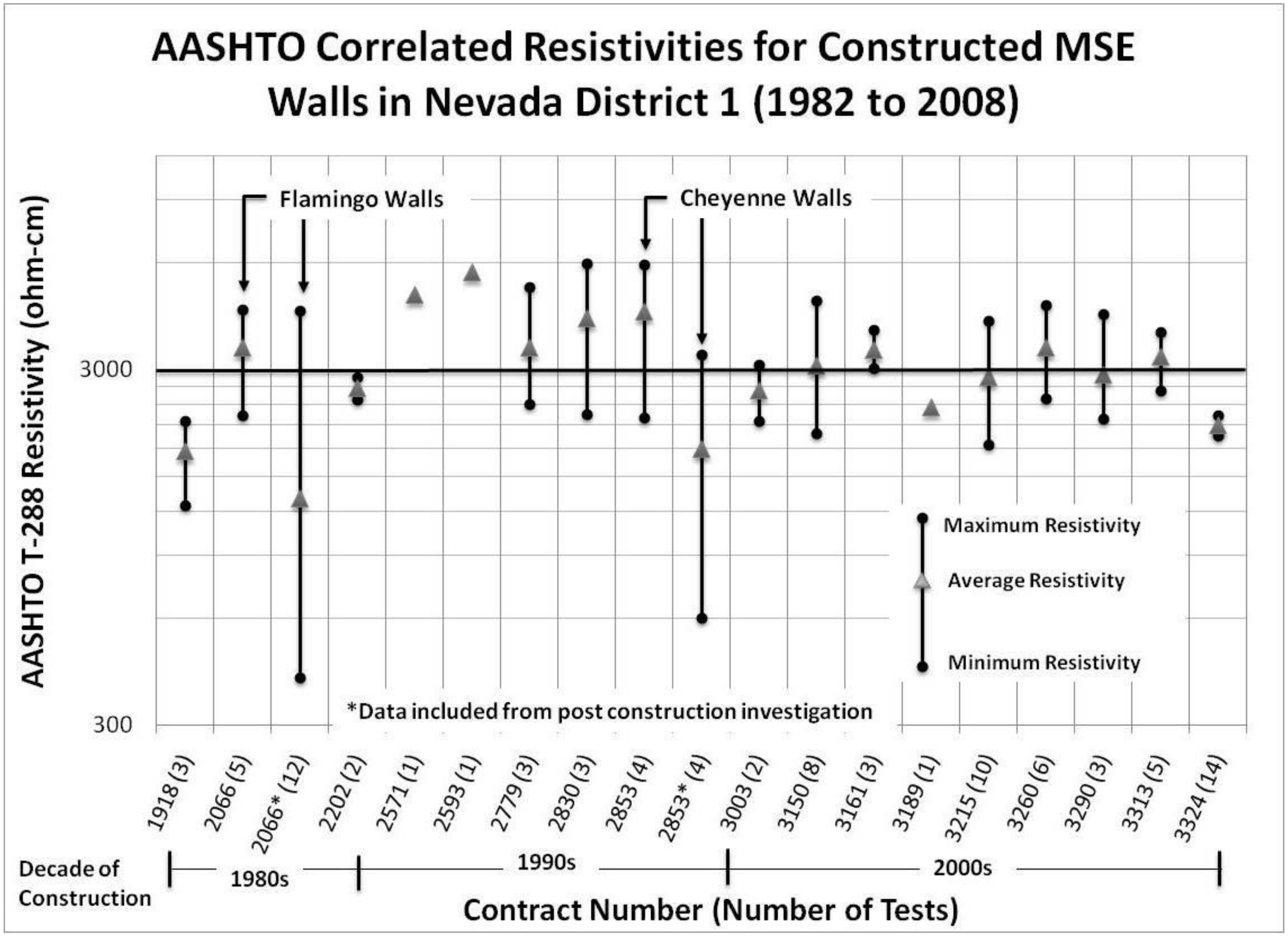

Figure 61. AASHTO Resistivity Data for MSE Wall Contracts in Nevada – District 1 Only

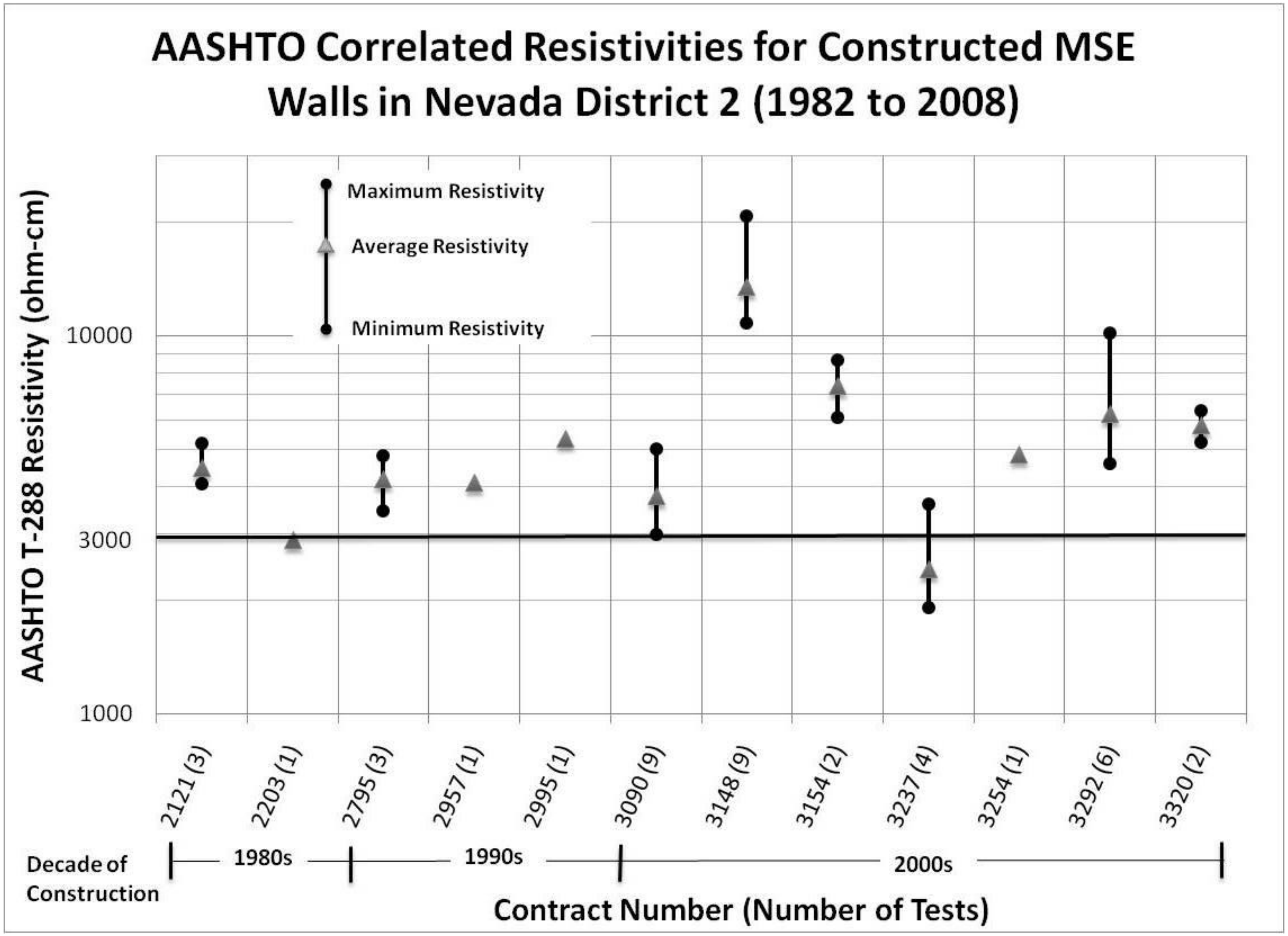

Figure 62. AASHTO Resistivity Data for MSE Wall Contracts in Nevada – District 2 Only

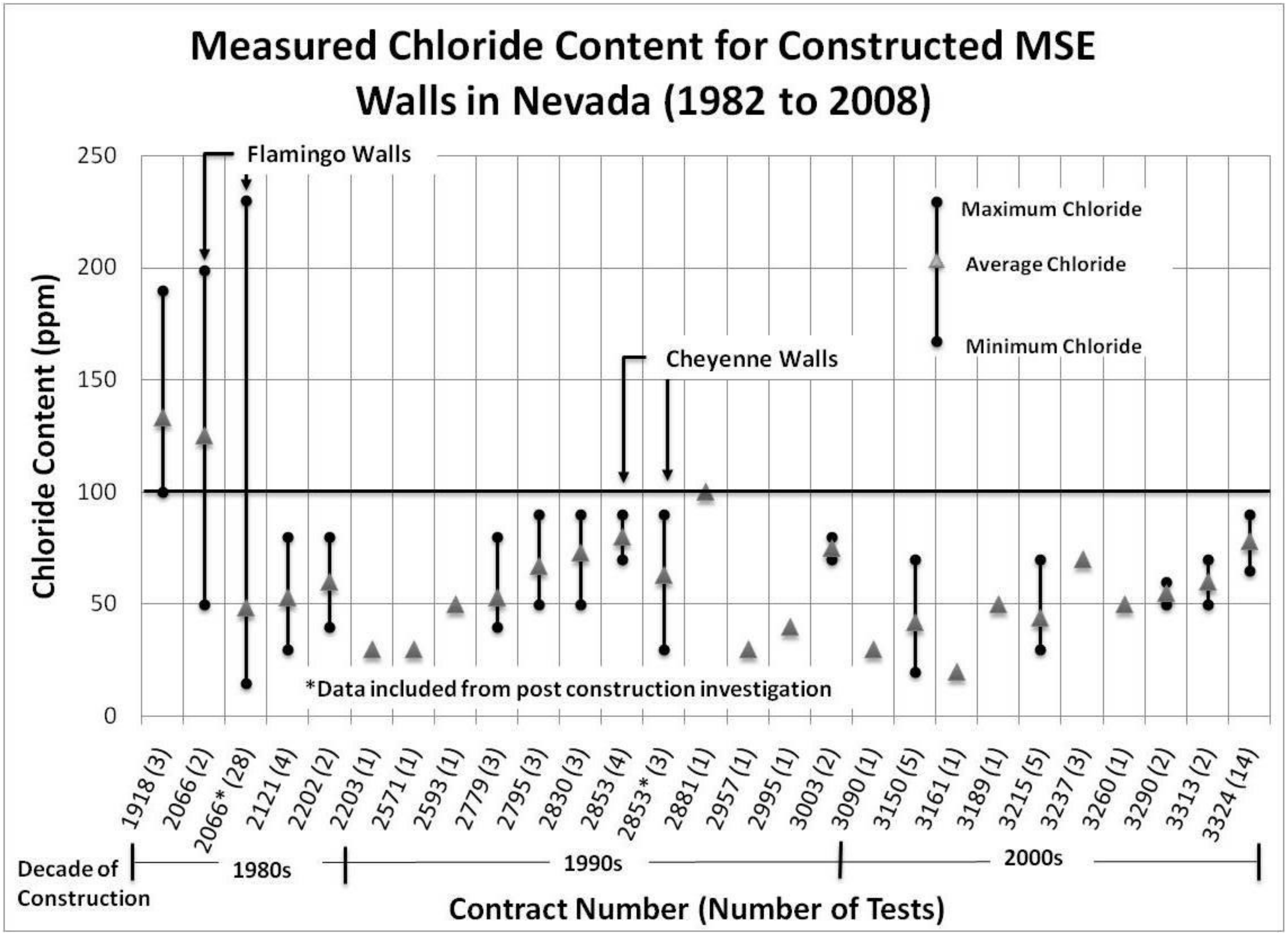

Figure 63. Chloride Data for MSE Wall Contracts in Nevada

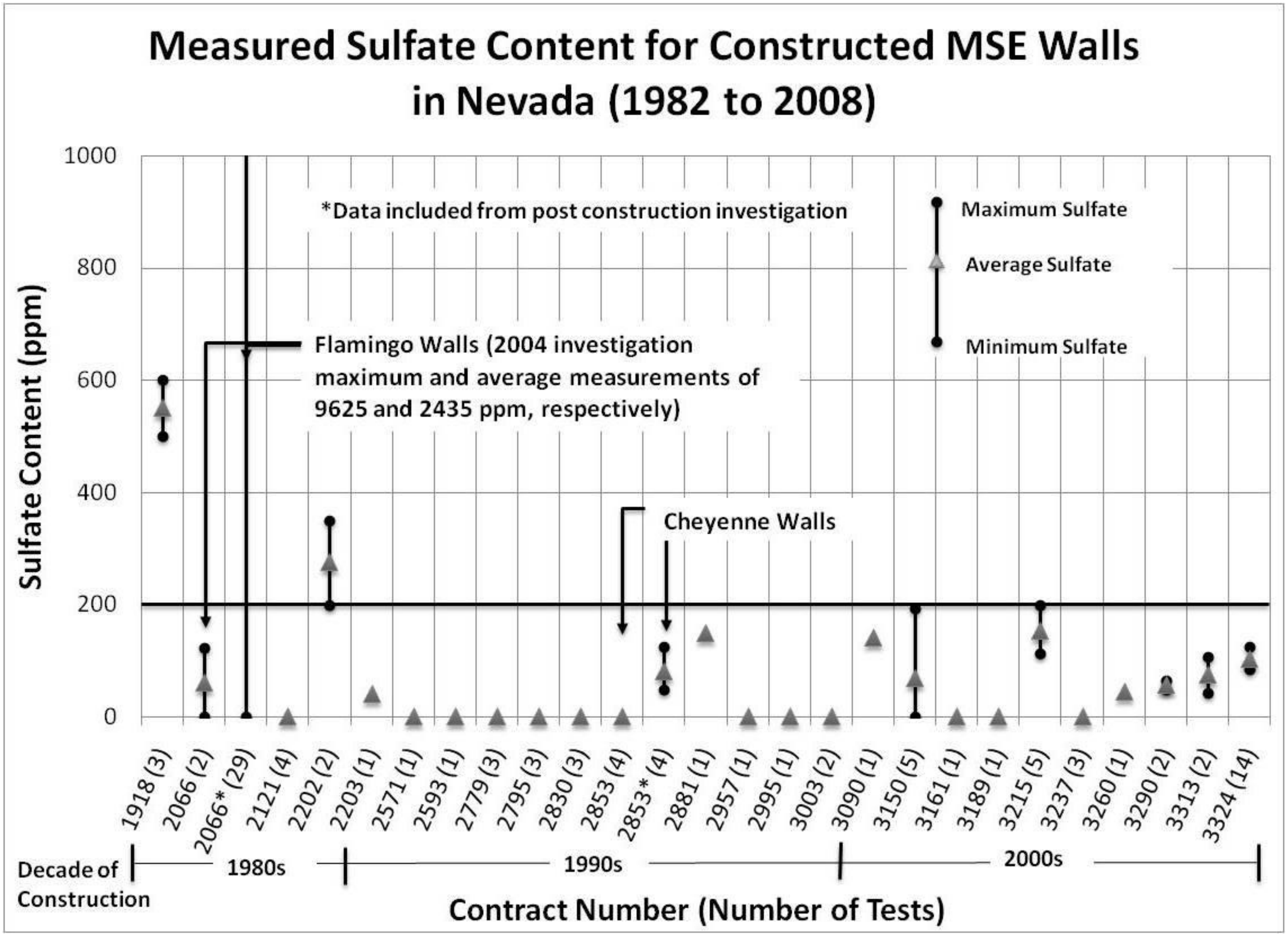

Figure 64. Sulfate Data for MSE Wall Contracts in Nevada

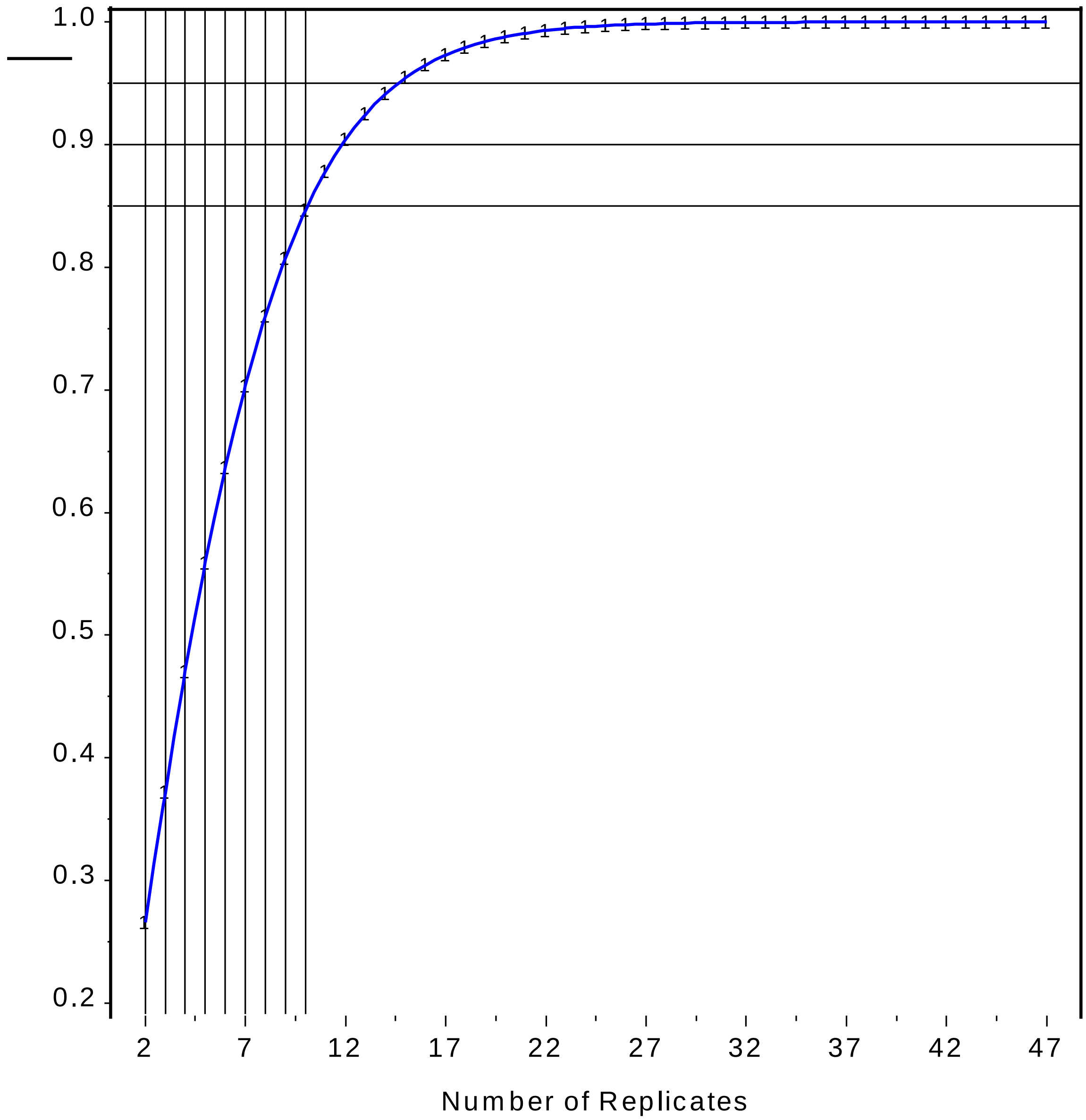

Figure 65. Power Analysis Results Identifying Minimum Statistically Significant Sample Size

Appendix A

A.1 Flamingo Wall Stability Calculations

The internal stability analysis for reinforcement tensile failure is based on the design calculations in the 2007 AASHTO LRFD Design manual (AASHTO 2007). Equation references to this AASHTO procedure are in parentheses. There are four parts to this analysis. The first is to calculate tensile loading under static conditions. The second is to calculate tensile loading under seismic conditions. Once the loads are calculated the tensile capacity of the reinforcements is calculated, based on the power loss models developed in Chapter 4. Finally, the capacity to demand ratio is calculated in order to evaluate stability. These steps are performed below for one grid level located in Flamingo Wall #2 at 23 feet from the top of the wall.

1. Calculate static loading:

   $T_{max} = \sigma_h * S_v = \gamma_p * \sigma_v * k_r * S_v = 2530$ lb/lf of wall (Equations 11.10.6.2.1-1 and 11.10.6.2.1-2)

   Where:

   $\gamma_p = 1.35$ (Table 3.4.1.2)

   $\sigma_v = \gamma_{soil} * h = 120pcf*23ft = 2,760psf$

   $k_r = k_a * k_r / k_a = 0.283*1.2$ (Figure 11.10.6.2.1-3)

   $S_v = 2ft$

2. Calculate seismic loading:

   $T_{total} = T_{max} + T_{md} = 2530 + 383 = 2914$ lb/lf of wall (Equation 11.10.7.2-2)

   Where:

   $T_{md} = \gamma_{EQ} * P_i * L_{ei} / \sum L_{ei} = 383$ lb/lf (Equation 11.10.7.2-1)

   $\gamma_{EQ} = 1.00$ (Table 3.4.1-1)

   $P_i = A_m * W_a = 4830$ lb/lf

   $A_m = (1.45 - A) * A = 0.195g$ (Assuming A = 0.15g)

   $W_a = 24,800$ lb/lf (Weight of active wedge)

   $L_{ei} = 14.6$ ft/lf

   $\sum L_{ei} = 184$ ft/lf

3. Calculate tensile capacity (calculated at 50 years using the average power loss model from Chapter 4 and a W9.5 soil reinforcement):

$T_{allow} = A_c*F_y/b = 2340$ lb/lf of wall (Equation 11.10.6.4.3a-1)

$A_c = 0.0209 in^2$ (Radial loss of 0.185 inches/side)

$F_y = 70$ ksi
$b = 0.625$ ft/bar

4. Calculate the capacity to demand ratio (C/D):

$C/D = \varphi*Tallow*R_c/T_{max} = 0.92$ (need C/D>1 for stability)

$\varphi = 1.00$ (full yield strength is used)
$R_c = 1.00$ (continuous coverage by WWF)

This C/D calculation can be repeated for the seismic case as well.

A.2 Seismic Acceleration Input Parameter

The latitude and longitude for the Flamingo walls in Las Vegas, Nevada are 36.115˚ and 115.083˚, respectively. Using the conservative assumption that the walls are on supporting soils that are classified as a Class D site, the USGS model estimates S_{DS} equal to 0.521g. To estimate a_{max} S_{DS} is divided by 2.5. This results in an input motion of 0.208g. Therefore, 0.21g is used in the seismic stability analysis in Chapter 4.

This methodology is based on seismic ground motion codes presented in ASCE 7-05 and the USGS Model can be found at http://earthquake.usgs.gov/research/hazmaps/design/.